STRENGTH AND VOLUME CHANGE CHARACTERISTICS OF A BITUMINOUS MIXTURE UNDER TRIAXIAL TESTING

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Joint Highway Research Project

PURDUE UNIVERSITY
LAFAYETTE INDIANA

Final Report

STRENGTH AND VOLUME CHANGE CHARACTERISTICS OF A BITUMINOUS MIXTURE UNDER TRIAXIAL TESTING

TO: K. B. Woods, Director
Joint Highway Research Project

August 3, 1960

FROM: H. L. Michael, Assistant Director
Joint Highway Research Project

File: 2-4-16
Project: C-36-6P

Attached is a final report titled, "Strength and Volume Change Characteristics of a Bituminous Mixture under Triaxial Testing". It has been prepared by Mr. James H. Schaub, under the direction of Professor W. H. Goetz. Mr. Schaub also used this report as his dissertation for the Ph.D. degree.

The report is presented to the Board for the record.

Respectfully submitted,

Harold L. Michael

Harold L. Michael, Secretary

HLM:lmc

Attachment

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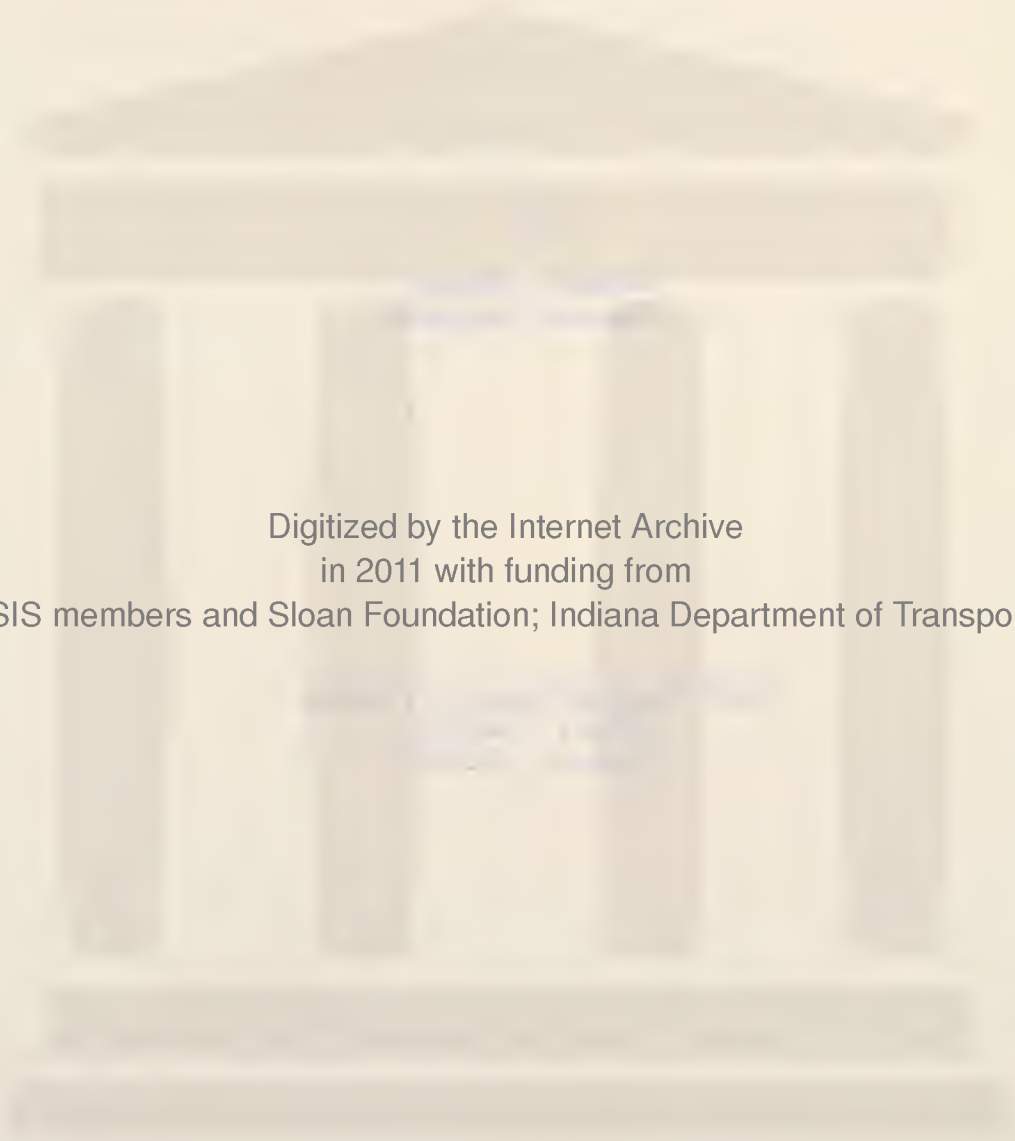

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ABSTRACT

Schaub, James Hamilton. Ph.D., Purdue University, August, 1960. Strength and Volume Change Characteristics of a Bituminous Mixture Under Triaxial Testing. Major Professor: William H. Goetz.

A laboratory study was performed upon a bituminous mixture using the constant-lateral-pressure triaxial test and incremental static loading. The study was initiated to establish whether or not bituminous mixtures changed volume during testing and, if so, the magnitude of the changes and their relationship to strength. The effect of various conditions of drainage permitted during a test upon observed strength values was established.

The study was accomplished using a mixture composed of a constant aggregate gradation and varying quantities of a single bituminous cement. The mixture was compacted into specimens having uniform density and asphalt content throughout the specimen by the use of the kneading compactor.

Equipment was developed to permit measurement of the changes of volume of a specimen subjected to triaxial loading during the performance of a test. The equipment measured the change in volume of the triaxial cell liquid which reflected changes in the volume of the test specimen.

The stress-strain and volume change characteristics of sixty-five specimens were established. All tests were conducted at confining

pressures of 15, 30, or 60 psi and axial loads were applied through a lever system.

The results show conclusively, for the particular mixtures and procedures used, that bituminous mixture specimens change volume during the performance of a triaxial test. All specimens tested showed a slight decrease in volume followed by an increase in volume to failure. The volume increase was linear with axial strain from a strain corresponding to approximately minimum volume to failure. The results of the volume change studies indicated that dilatancy occurs in bituminous mixtures in much the same fashion as in dense granular soils.

Volume changes were analyzed in terms of the changes of percent voids and void ratios of the specimens. For the bituminous mixtures tested, a relationship between void ratio and percent voids of a specimen, at two percent strain and at the Proportional Limit, and the deviator stress at these points was established. This relationship was found to be linear between void ratio or percent voids (arithmetic scale) and deviator stress (logarithmic scale) and was dependent upon the initial void characteristics of the specimen. The relationship was apparently independent of confining pressure. To verify the unique character of the voids--deviator stress relationship, specimens were compacted to a given set of initial conditions and these conditions changed by water-saturation of the specimens prior to testing.

It was established, for the mixtures and procedures used, that the drainage conditions existing during a test had no measurable effect on the magnitude of observed shear strength values. Indirect evidence did indicate, however, that negative pore pressures developed during test-

ing. As the negative pore pressures did not affect the observed strength results, it was concluded that the effect of these pressures was so small as to be masked by the high strength values of the mixtures tested.

The effect of asphalt content and compaction pressure on the value of apparent cohesion and apparent angle of internal friction was investigated. The results clearly indicate that both of the apparent strength parameters were affected not only by asphalt content but by compaction pressure as well.

Results show that the aggregate density variation of the mixture used provided a fairly good measure of the relative values of observed compressive strengths while bulk density did not.

Data were gathered on the effect of asphalt content and compaction pressure upon the Modulus of Deformation of the mixtures studied. In general, it was established that the Modulus of Deformation was affected greatly by asphalt content at high compaction pressures but only slightly at low compaction pressures.

A limited study of the Deformation Ratio of the particular bituminous mixture used was accomplished. Deformation Ratio was defined as the ratio of lateral strain to axial strain and is analogous to Poisson's Ratio for elastic materials. The results of the study indicate that the Deformation Ratio varies from values of less than 0.5 to nearly 1.0 during a test. Further, it was shown that the apparent angle of shearing resistance was mobilized gradually with a continuing increase in Deformation Ratio to a point at which the maximum apparent angle of shearing resistance was developed. The maximum apparent angle

of shearing resistance established by the Deformation Ratio studies compared favorably with the same measure determined by the Mohr circle analysis of stress at failure.

INTRODUCTION

For approximately twenty-five years the attention of paving design engineers has been directed in part towards the use of a triaxial shear strength method of test to evaluate the properties of bituminous mixtures. Interest in this form of test, particularly as a means of evaluating fundamental strength properties of bituminous mixtures, has become increasingly great. At the present time there are several techniques in use that utilize the principles of triaxial shear testing for the design and strength evaluation of bituminous concrete mixtures.

The value of the triaxial shear test as a research tool is generally accepted; however, the utilization of this method of test for design purposes is limited. In comparison to other methods of test, the triaxial shear strength procedure attempts to evaluate the fundamental strength properties of a mixture under specified and known loading conditions. The stress system that acts upon a sample during the triaxial test approximates generally the system of stresses that are present during the loading of a mixture when it acts as a part of a pavement structure. Furthermore, the triaxial test makes use of a strength theory that agrees reasonably well with experimental results.

The problem of determining the strength characteristics of a material is of major interest in the study of all road-building materials. The field of soil mechanics has made the greatest use of triaxial shear testing for this purpose to the present time. This field of study has

developed a working explanation of the variation of measured shear strength with test procedure and with prior stress history for saturated soils. More recently, soil mechanics investigators have devoted their attention to the problem of establishing the shear strength of partially saturated soils. On the basis of the work done in soil mechanics, several general statements may be made regarding the shear strength of soils.

1. The shearing resistance of saturated, remolded cohesive soils is a function of the effective normal stress and the void ratio at failure.
2. The void content at failure of a partially saturated soil bears a unique relationship to the deviator stress at failure for a given set of initial conditions.
3. The previous two statements indicate that observed shear strength values are directly affected by the method of performance of the triaxial test and by the void content at failure.

An apparent similarity between a partially saturated soil, a three phase system composed of soil solids, water and air, and a bituminous mixture, composed of aggregate solids, bitumen, and air serves as a basis for an attempt to investigate the application of triaxial shear strength concepts developed in soil mechanics to the study of the strength of bituminous mixtures. Present practice in the triaxial testing of bituminous mixtures does not consider the effect of the method of performance of the test on observed shear strength values. Though air voids are of major interest in bituminous mixture design, little or no

attention has been devoted to the evaluation of the void content of bituminous mixtures at failure criteria.

It appeared desirable to establish whether or not the method of performance of a triaxial test on bituminous mixtures affects the observed shear strength of the mixture. In addition, a knowledge of the magnitude of change, if any, in the void content of a mixture during shear would aid in establishing a proper perspective of the role of air voids in the design of an adequate bituminous mixture for road use.

The study being reported represents an attempt to increase the fund of knowledge of the action of bituminous mixtures under triaxial loading. The specific purposes of the study were as follows:

1. To develop test equipment and procedures that would permit the study of the affect of the method of test on observed shear strength values and further permit the evaluation of the void content at any point during the progress of a test.
2. To develop a method of fabricating a sample suitable for triaxial testing that is reasonably uniform throughout with respect to density and asphalt content.
3. To establish whether the method of test procedure affects the observed shear strength of a bituminous mixture and the magnitude of this effect if any.
4. To establish whether there is a change in the volume of a bituminous mixture subjected to a

triaxial stress system and, if so, to establish the significant change in voids that is reflected by a change in observed shear strength.

5. To investigate the hypothesis that there is a unique relationship between the void content at failure and the shear strength of a bituminous mixture prepared to a given set of initial conditions.

The current status of knowledge of the triaxial shear strength of bituminous mixtures was established by a review of the available literature. The equipment developed for the study is presented. The tests used in an effort to satisfy the purposes of the study are reported and the results of these tests are discussed. Suggestions for research that would provide a logical continuation of this study are presented.

REVIEW OF LITERATURE

Numerous test methods have been developed and used with varying degrees of success for the design of bituminous mixtures. In all of these procedures, a measure of the stability of the mixture is obtained. Neppe (1) has classified the various stability tests as follows:

1. Compression
    a. Unconfined compression including immersion compression
    b. Constrained compression triaxial shear tests including both open and closed systems
2. Tensile and elongation tests
3. Bending tests
4. Shear tests
5. Extrusion tests
6. Indentation and penetration tests
7. Impact and toughness tests
8. Revolving drum brittleness tests
9. Punching shear tests
10. Vibration resistance tests
11. Abrasion tests
12. Load transmission tests including road machine and track service tests

By far the majority of stability tests performed on bituminous mixtures in the USA would be classified as compression tests by the above system.

In the continuing effort to develop an understanding of the mechanisms by which a bituminous mixture supports load and by which failure occurs, more and more attention has been directed to the triaxial compression test. In the opinion of numerous investigators the triaxial test is the only test procedure that offers the possibility of measuring stability in such a way that the stress system is subject to a rational analysis at all times during the test and also provides a stress system similar to that to which the mixture will be subjected in service. In illustration of this opinion, Monismith and Vallerga (2) state: "Stability is best defined as a load to cause a certain amount of deformation, said deformation depending upon expected field conditions, and that only a form of triaxial compression test will properly measure this property." Smith (3) has written: "In this test (triaxial) method the stresses acting upon the specimen during testing closely approach the system of stresses existing in a flexible pavement or pavement foundation when they are supporting a load. Triaxial shear theories are in close agreement with actual experiment." In reference to the stability of one-size aggregate mixtures, Oppenlander (4) states: "Excluding empirical test methods, the triaxial compression test is the only suitable method for the fundamental evaluation of the true stability of this bituminous mixture having a one-size aggregate."

The purposes of this study are basically an effort to further advance the fund of knowledge concerning the action of bituminous mixtures

under triaxial loading in the belief that the foregoing statements correctly present the value of this type of test.

The literature review for the study required the perusal of numerous articles covering a wide range of topics relating in some way to the purposes of the study. For simplicity's sake, the literature review is presented in three parts:

1. Triaxial Testing of Soils,
2. Triaxial Testing of Bituminous Mixtures,
3. Relationship of Air Voids to Mixture Design.

Triaxial Testing of Soils

Expressions for the shearing resistance of earth masses have been based historically upon the hypothesis of Coulomb (5) which was first advanced in 1776. This hypothesis may be summarized algebraically in modern terminology as follows:

$$s = c + \sigma \tan \phi \qquad \text{Equation 1}$$

in which s is the shearing resistance of the material; c is the unit cohesion; σ is the value of the normal stress on any portion of a plane in the material; and ϕ denotes the angle of internal friction of the material. This expression of shearing resistance considers both c and ϕ to be constant for any given material. The Coulomb equation and the accompanying concept of constant shear-strength parameters have influenced the expression of shearing resistance of soils since its elucidation.

Beginning in 1925, the work of Dr. Karl Terzaghi called the attention of engineers to the influence of pore water pressure and the effect

of this pressure on the results of shear tests. In 1936, Terzaghi (6) pointed out that the σ term of the Coulomb equation must be considered as the effective normal stress (total normal stress less pore water pressure) on the plane of failure. Further, the Terzaghi Theory of Consolidation (7) implies that the magnitude of pore water pressure developed as a result of loading is directly affected by the amount and rate of drainage permitted during the test. This implication was borne out by the subsequent investigations of Jurgenson (8) and others at the Massachusetts Institute of Technology.

The concepts of the shear strength of soils were drastically revised as a result of the independent investigations of Hvorslev (9) and Rendulic (10). The conclusions of these investigations include the following:

1. During shear a cohesive soil will tend to expand or to decrease in volume depending upon the state of consolidation of the specimen.
2. The pore-water pressure will be equal to atmospheric pressure at all times if the rate of load application is sufficiently slow so that the void ratio of the soil may become fully adjusted to the prevailing stress conditions at any given time.
3. Under conditions of stress application that is so rapid that full adjustment of the void ratio cannot be accomplished, the pore pressures will be in excess of atmospheric pressure if the sample tends to decrease in volume. If the sample tends to increase in volume under similar

stress application conditions, the pore pressure will be lower than atmospheric pressure.

4. The shearing resistance of saturated, remolded cohesive soils is a function of the effective normal stress and the void ratio or water content at the moment of failure.

5. The true angle of internal friction of such soils is a constant; however, the cohesion of the soil is a function of the void ratio at failure.

On the basis of the Hvorslev investigations, the Coulomb equation may be written in a more fundamental form for saturated soils:

$$s = c_e + (\sigma - u) \tan \phi_e \qquad \text{Equation 2}$$

where c_e is the true cohesion of the material, ϕ_e is the true angle of internal friction and is essentially a constant, and u is the value of the pore-water pressure or neutral stress.

The validity of the Hvorslev concepts has been verified by numerous investigations in Europe (11)(12)(13). Skempton and Bishop (11) state unequivocally, "In the authors' opinion it (the Coulomb-Hvorslev equation) must form the basis of any fundamental considerations of shear strength."

In the practical determination of the shear strength of a soil the application of the Hvorslev concepts of true cohesion and true angle of internal friction involves the measurement of pore pressures developed during the test. In the interest of simplicity and reliability the use of

test values of c and ϕ determined under test conditions similar to those expected in the full-scale structure are often used in lieu of the more correct c_e and ϕ_e values. Rutledge (14) credits A. Casagrande with the proposal for the following three basic types of tests which limit the conditions of pore pressure and intergranular stress that is acting and establish stress systems similar to expected field conditions:

> "a. Quick tests (Q). In these tests no drainage of pore water from the specimen is permitted. Neglecting any possible effects of axial deformation on a tendency toward volume change in the soil, any applied stress creates an equal pressure in the pore water. The simple unconfined compression tests (U) properly performed on saturated clays are tests of this type.
>
> "b. Consolidated-quick (Q_c). In these tests complete consolidation, under an applied pressure that is equal in all directions or hydrostatic, is permitted. The axial stress is increased to failure without further drainage of pore water. Again neglecting effects of axial deformation on tendency toward volume change, the pore pressure is equal to the axial deviator stress, in other words the axial stress that is added to the initial conditions of externally applied hydrostatic pressure.
>
> "c. Slow tests (S). In these tests complete drainage of pore water is permitted under both the applied hydrostatic pressure and the subsequent increase in axial stress. In a perfect slow test the pore water pressure is equal to zero throughout the test."

All of the concepts stated in the preceding discussion are applicable to saturated clays. The test procedures may be applied to either direct shear tests or to triaxial compression tests. For reasons of better test control and more rational test conditions the triaxial compression test is preferred for the shear testing of soils. Rutledge (14) has provided the following complete definition of triaxial compression:

> "Triaxial Compression designates a condition of test in which stresses of known magnitudes in three rectangular directions are applied to a specimen that is free to change shape in three directions. The stresses may be equal in three directions, equal in two directions, or unequal in all three rectangular directions. The only required condition is that the stresses cause a shortening of the specimen in the direction of its axis. The common conditions of stress in triaxial compression are equal stresses in three coordinate directions, called triaxial consolidation because the stresses cause only volume decrease in the specimen, and equal stresses in two lateral directions with a greater stress in the axial direction. The latter case is the stress condition commonly known as triaxial compression and, in general, it causes both volume changes and shear deformations in the specimen."

In simple terms the triaxial compression test may be considered as a compression test in which the specimen is supported throughout its length by equal stresses acting in all lateral directions.

In summary, the knowledge regarding the shear strength of saturated clays is well expressed by Rutledge (14): "The maximum principal stress difference ($\sigma_1 - \sigma_3$), usually called the compressive strength of clay, depends only on water content at maximum stress, provided the test specimens have not been preconsolidated under pressures greater than the test maximum stress and then allowed to rebound to equilibrium under lower pressures." It should be noted that a similar and equally valid statement may be made using void ratio as a reference rather than water content as these characteristics are proportional to each other for a saturated soil.

With regard to the strength of cohesionless soils, Rutledge (14) reports that the strength is dependent upon the angle of internal friction of the material which is a function of soil type and the density of the specimen at the time of test but is independent of the minor principal

stress and method of test. The effects of drainage during testing of cohesionless soils are clearly established by Casagrande (15) who called attention to the fact that cohesionless soils tend to change volume during shear. Whether or not the volume change is expansion or a reduction in volume is dependent upon the density of the specimen. Loose specimens tend to be reduced in volume while dense specimens tend to expand. If such a material is saturated, the tendency to change volume effects pore water pressures and consequently the strength of the material. The void ratio of a soil at which neither expansion or reduction in volume occurs during shear was given the name 'critical void ratio' by Casagrande.

The state of knowledge concerning the shear strength of partially saturated soils is not so well established as that for saturated materials. Rutledge (14) reports:

> "Four major variables effect the strength of a soil in this group. These are: the minor principal stress, the dry soil density, the water content, and the percent of voids filled with water, usually called the degree of saturation. The last two variables are not independent. ...In addition, for compacted soils the method of and conditions during compaction seem to have independent effects on test strength."

A recent study of the compression characteristics of partially saturated soils by Yoshima (16) makes the following statement: "In contrast to the saturated system, an important feature of the partially saturated system is the fact that the compressibility of air allows immediate compression of the spring (soil skeleton) without the need of displacing fluids out of the system." A similar result was reported in a comprehensive study of the shear strength of partially saturated soils by Leonards (17) as follows: "...during the application of

shearing stresses on partially saturated clays, changes in void ratio occur that are not accompanied by corresponding changes in water content. The nature of these changes in void ratio depends on the initial conditions, the confining pressures, the preconsolidation pressure, and the quantity of water added to the soil from an external source."

The change in volume of partially saturated soils without change in water content during triaxial shear has been noted by Holtz (18), Taylor (19), Wagner (20) and others. Holtz explains this action as follows: "The void spaces of a soil mass are filled with a partially compressible fluid which consists of a mixture of gas and water, the compressibility of the mixture being dependent upon the relative proportions of gas and water. Loading a specimen in shear produces volume changes which must be accompanied by an extrusion of the pore fluid from the soil mass or by compression of the fluid."

Wilson (21) points out that the air in the voids of a partially saturated soil acts in the same fashion as water in reducing the effective stress developed under load. "The distribution between effective and neutral stress upon loading depends not only on the original degree of compaction achieved, but on changes in void ratio and degrees of saturation that may develop in the course of time. Therefore, the strength properties of a compacted mass will differ at different elapsed times after compaction." These comments would indicate that it is possible for the strength of a compacted material to vary with time even though no water content changes occur.

With the consideration of the variables involved in the shear strength of a partially saturated soil it is apparent that no simple

relationship can predict the strength characteristics of such materials. Rutledge (14) noted that the following three test characteristics exist for partially saturated soils:

"a. In Q-tests the principal stress difference, or compressive strength, increases with lateral pressure.

"b. Very little difference in strength is observed in the results of Q- and Q_c-tests.

"c. The relation between compressive strength and water content at the end of test varies with the minor principal stress."

Leonards (17) concludes that for a given set of initial conditions,

> "...the pattern of strength variation for partially saturated, compacted clays can be represented by a relationship between the void ratio at failure (plotted to an arithmetic scale) and the maximum value of the principal stress difference, or compressive strength (plotted to a logarithmic scale). For a specified set of initial conditions this relationship is independent of the confining pressure, the amount of drainage permitted, the water content, and the degree of saturation. It is indicated that the relationship is unique even if the degree of saturation (or water content) is increased by the addition of water from an external source, and there is a possibility that this uniqueness will exist even after the soil has been preconsolidated and then allowed to rebound fully at a lower confining pressure."

The analysis of the results of a triaxial compression test commonly is accomplished by the application of the Mohr Theory of Failure (22). In general, the Mohr theory hypothesizes that the limiting shear stress depends on the normal stress acting on the plane of failure and that failure depends on the maximum magnitude of the difference in principal stresses acting assuming that the intermediate principal stress does not effect the shear strength. Graphically, the Mohr theory is expressed as a locus of points (called the envelope of failure) whose coordinates are the values of normal and tangential components of stress

on the plane of failure resulting from paired values of major and minor principal stresses occurring at a point of incipient failure. A circle constructed with the maximum difference in principal stresses as a diameter is tangent to the envelope of failure and represents the stress components on all planes through a point in the stressed mass.

Cohesionless soils generally exhibit a straight line envelope of failure though a curvature of the failure envelope may be observed at high confining pressures. Normally consolidated, saturated cohesive soils exhibit a straight line envelope of failure while pre-consolidated, saturated cohesive soils display a curved envelope for confining pressures less than the pre-consolidation pressure. The failure envelope is a graphical representation of the Coulomb equation. The slope of the envelope defines the angle of internal friction while the intercept of the envelope with the vertical axis establishes the cohesion value. The parameters established by this analysis are apparent values if total stresses are used to determine the envelope of failure. The apparent parameters are a function of the drainage conditions permitted during the test or of the effective stresses acting during shear. The true angle of internal friction and true cohesion as used in the Hvorslev modification of the Coulomb equation (see Equation 2) may be defined using samples of a material in both the normally consolidated and pre-consolidated states that have the same void ratio at failure.

In the case of partially saturated cohesive soils, the Mohr envelope frequently shows a curvature that is concave downward for an undrained or Q-test. Bishop and Henkel (23) explain this curvature as follows: "The deviator stress at failure is found to increase with cell pressure. This increase becomes progressively smaller as the air in

voids is compressed and passes into solution, and ceases when the stresses are large enough to cause full saturation." For the consolidated-undrained or Q_c-test, the curved envelope will develop throughout the lateral stress range below that required to compress all the air voids or to develop complete saturation. The curved envelope of failure is not associated with the drained or S-test due to the complete drainage that is permitted under all applied stresses. In all cases an envelope of failure prepared using effective stresses may be constructed that will permit the definition of the true shear strength parameters.

Triaxial Testing of Bituminous Mixtures

Partially saturated soils and bituminous mixtures are analogous three-phase systems. The former consists of a solid phase of soil particles, a liquid phase composed of water, and a gaseous phase that is the air in the unfilled voids. Bituminous mixtures are composed of a solid phase which consists again of solid particles or the aggregate, a liquid phase formed of the bitumen, and a gaseous phase consisting of the air in the air voids of the mixture. Because of the striking analogy between these two widely used construction materials, it seems to be a logical step to attempt to analyze the shearing resistance characteristics of bituminous mixtures in light of the available knowledge and methods established in soil mechanics.

The desirability of using the triaxial test for the analysis of the strength characteristics of road materials was voiced by Palmer (24) in 1938: "It was the concensus of this meeting (Eighteenth Annual Meeting, Highway Research Board) that the triaxial compression or stabilometer device is the most useful shearing test method and despite all obstacles it is proposed to obtain and use complete stress-deformation diagrams in connection with highway problems." The asphalt paving

industry has been aware of the potential offered by the triaxial method of test for many years. As early as 1934, Stanton and Hveem (25) made mention of the early forms of the stabilometer and a test procedure that involved supporting a specimen laterally while deforming it by the application of an axial load.

Beyond the logical extension of the knowledge of the shear strength of soils to the similar bituminous concrete systems, the triaxial method of test offers other advantages for the study of the strength of bituminous mixtures. Smith (3), in discussing the triaxial test method has stated, "In this method the stresses acting upon the specimen during testing closely approach the system of stresses existing in a flexible pavement or pavement foundation when they are supporting a load. Triaxial shear theories are in close agreement with actual experiment." Endersby (26) pointed out:

> "A body of material in a pavement is subjected to a combination of vertical load and lateral restraint. ...experimental means of measuring the effect of this combination are called for. In addition, an analysis of the respective functions of binder and aggregate require a method which shows the cohesion and friction as separate forces. All these conditions are fulfilled by the triaxial test when combined with the Mohr diagram type of representation of results."

A further advantage of the triaxial test for evaluating the shearing resistance of bituminous mixtures is the ease with which the procedure may be adapted to the particular needs of research. Goetz (27) has summarized this advantage as follows: "As a result of this rather extensive investigation, we favor the use of the triaxial test method as a research tool for the evaluation of mix variables, and as a basis on which to obtain data to extend present knowledge of bituminous

pavement design." The advantages of the triaxial test for research studies were reiterated by Goetz and Schaub (28) by emphasizing the versatility of the triaxial method of test in the investigation of the numerous variables in bituminous mixture design.

Triaxial testing methods for bituminous mixtures have developed along two similar but distinctly different lines. The two general procedures often are called the 'closed' system and the 'open' system. The open system is similar in form to the tests used for soils as previously discussed. A lateral pressure is applied to the specimen and held constant while an axial load is applied to cause failure of the specimen. In contrast, the closed system permits the lateral pressure to vary as a result of the tendency of the sample to deform under an applied load. It is likely that Hogentogler (29) first introduced these terms in defining various methods of triaxial test during a summary of triaxial shear tests presented in 1939. Hogentogler's classification states: "Methods employed included a 'closed system' which prevents volume change of samples, and an 'open system' which permits swell or consolidation during test. An impervious casement which prevents entrance or escape of air and water encloses samples in the closed system and placing them between porous stones provides for the entrance or egress of air and water in the open system." The primary advantage claimed for the closed system is that a value of c and ϕ for a mixture may be established from a test performed on only one sample whereas a minimum of two and preferably three or more samples, tested at different lateral pressures, are required for this purpose when using the closed system. For simplicity's sake it seems desirable to refer to the

open-system test as a constant-lateral-pressure test and to the closed system as a variable-lateral-pressure procedure.

A second major divergence in the test procedures presently in use for testing bituminous mixtures is the relationship between the height (H) and the diameter (D) of the specimen tested. One group of test procedures, notably those represented by the Stabilometer method, utilizes a sample in which the height is less than the diameter of the specimen. The adherents of this technique argue that such a sample is similar to the prototype existing in the roadway and, therefore, test results may be considered as representative of pavement performance. Other triaxial tests are performed using a sample in which the height is equal to or greater than the diameter. In such samples, shear planes are free to develop within the sample and a theoretical analysis is valid. A sample with a height that is small relative to its diameter will have confining stresses induced at the ends of the specimen by the loading heads. Rutledge (14) notes that samples with an H/D ratio of two to three are satisfactory for soils. H/D ratios greater than that noted may not be representative due to the possible introduction of column action in the sample. The effects of a low H/D ratio have been discussed by Pheiffer (30), Smith (3) and others.

Unless otherwise noted all references to triaxial testing in this discussion refer to samples with an H/D value of approximately two and to an open-system or constant-lateral-pressure method of test.

Effect of the Binder

The analysis of the results of triaxial tests of bituminous mixtures is complicated by the nature of the binder and by the relationship

between the binder and the aggregate component. Lee and Markwick (31) have stated: "From the point of view of their mechanical properties bituminous materials exhibit the characteristics of (1) plastic materials, which flow under sustained load, (2) elastic materials, which show pronounced hardness and elasticity and characteristics of solids, (3) granular materials characterized by (a) void content and porosity, (b) consolidation, (c) dilatancy when subjected to deformation."

The observed shear strength of a bituminous mixture established by triaxial testing is affected by the viscosity of the binder. Nijboer (32) considers this phenomena as follows:

> "When using this test to the fullest extent the resistance to shear of the material under conditions prevailing in road work can be described by the three following physical magnitudes: (1) a frictional resistance, due to friction between the aggregate particles, which exert a pressure on each other (grain pressures) and is proportional to this pressure. The angle of internal friction is used as its characteristic in accordance with soil mechanics. (2) An internal resistance, a resistance the material is able to develop in the absence of frictional resistance. (3) A viscous resistance (in poises), developing only when the material is sheared off at a certain rate. This resistance is due to the presence in the material of the viscous asphalt cement. Attention should be drawn to the viscous resistance that even at low rates of deformation can acquire high values."

With the effect of the viscosity of the binder entering the concepts of shear strength of bituminous mixtures so profoundly, it is obvious that any variable influencing viscosity will also effect shear-strength measurements. The effect of rate of load application has received a great deal of study and comment. Goetz, McLaughlin, and Wood (33) have shown that the maximum compressive strength increases with increasing rates of load application. Goetz (27) has found that

cohesion parameter is drastically increased with an increase in testing speed but that the friction parameter is essentially constant over a wide range of testing speeds. Smith (3) suggests that a test procedure that eliminates the viscosity effects resulting from the rate of load application is desirable. "Application of static loads in increments is essential in order to eliminate the viscous resistance of the asphalt binder which is directly proportional to the rate of strain. It has been well established that static loads represent the severest stress conditions imposed upon bituminous pavement because under continuous stress conditions the viscous resistance of the asphalt or similar substances is essentially zero." Endersby (26) reached the same conclusion. Stevens (34) goes further and states: "The stability component resulting from the viscosity of the asphalt in a mix cannot be relied on as it becomes small or negligible under high temperatures and slow moving traffic or static load conditions, such as are experienced on city streets and airfield taxi-ways." The complexity of the problem of rate of load application and stability is summed very well by Stevens (34):

> "How much of the viscous resistance of the asphalt should be relied upon in judging the long time stability properties of an asphalt mix? It is true that it contributes to the stability of pavements under moving loads, and it is also true that a majority of the loads are moving. However, can one count on loads to be moving, or should one design for the worst condition, the static load, and use the viscous resistance of the asphalt as an added safety factor?"

In a similar fashion, variations in the temperature of the specimen at the time of testing affect the viscosity of the binder and so influence the observed shear strength of the mixture. Smith (3) has

written: "...the resistance of an asphaltic binder to imposed strains is dependent upon the consistency (viscosity) of the binder which in turn is affected markedly by change in temperature." The effects of test temperature on maximum compressive stress have been indicated by Goetz, McLaughlin, and Wood (33). In general, the maximum compressive stress observed is greater for lower temperatures for any given rate of deformation. Neppe (35) has found: "The mechanical stability of a bituminous mixture at any temperature, is approximately a direct function of log viscosity of the contained binder at that temperature and is independent of the source, nature, and properties of the latter constituent." Wood and Goetz (36) and McLaughlin (37) studied the relationships existing between compressive strength, temperature and rate of deformation and have established that for the mixtures and conditions studied the relationship could be expressed mathematically. In the opinion of Endersby (26) a test temperature of 77°F is a reasonable testing temperature as this value probably represents a good year round average and serves to bring out the role of the binder more clearly than a higher test temperature.

Influence of Sample Preparation

An additional factor influencing the observed strength values of a bituminous mixture is the method by which the sample is formed. Endersby and Vallerga (38) state: "The method of forming laboratory test specimens, even apart from corresponding bulk density, has a profound effect on test results, regardless of the test method or its validity." Basically all of the schemes in use for fabricating samples fall into one of three categories: (1) dynamic compaction as illustrated by the

compaction procedure utilized in the Marshall test (39); (2) static compaction which is one of the established procedures for preparing samples for use with the Smith Triaxial Cell procedure (40); and (3) kneading compaction which includes the procedure advocated by the adherents of the Stabilometer method of design (41), as well as the gyratory shear procedure (42) which is presently receiving extensive study.

The question of which compaction procedure is best suited for preparing samples for testing has received continuing study and comment. As the many design procedures in use are essentially empirical, the compaction procedure utilized should be the one that satisfies the correlation that forms the basis for the development of the particular design procedure. There is evidence, however, that different compaction procedures produce different particle orientations. It would seem reasonable to strive for a compaction procedure that produces a particle orientation similar to that obtained by compaction under field conditions. Extensive studies by a group of materials engineers on the Pacific Coast, generally known as the Triaxial Institute, investigated the kneading compaction procedure. Endersby (43) presents as one of the results of these studies: "...only kneading or rolling methods are likely to reproduce road conditions." Investigations by Vallerga (44), Hveem and Vallerga (45), Ortolani and Sandberg (42), and the experience of several agencies with the kneading compactors tend to substantiate this conclusion. The experience gained in preliminary studies of compaction procedures for this investigation led to the conclusion that samples of the desired height and having a reasonably uniform density

could be best obtained by using a kneading compactor. (See "Materials and Procedures".)

Criteria for Strength Evaluation

Still another basic factor which must be considered in a study of the characteristics of bituminous mixtures is the criterion to be used to judge the strength or 'stability' of the mixture. The plastic characteristics of bituminous mixtures permit such materials to accept large loads without shear failures but with high deformations. For design purposes, the stability of the mixture must be chosen with a concept of some limiting value of deformation in mind. Stanton and Hveem (25) define stability as "that property of bituminous pavements which tends to resist plastic deformation". Monismith and Vallerga (2) consider: "Stability is best defined as a load to cause a certain amount of deformation, said deformation depending upon expected field conditions...." For the purposes of this study, stability is considered as the difference between the major and minor principal stresses causing a given deformation of a specimen in a triaxial test. Compressive strength is considered as the maximum difference between the principal stresses acting at failure.

For any given compaction process and a fixed aggregate blend there is a variation in the density of the compacted specimen with variation in asphalt content. In general, as asphalt content increases there is an increase in density to a maximum value followed by a decrease in density with further increases in asphalt content. Lee and Markwick (31) explain the development of maximum density with increasing asphalt contents as follows:

> "The volume occupied by a consolidated mixture composed of a given quantity of aggregate with varying amounts of binder will be determined by the binder content, other factors remaining constant. Small percentages of binder act as a lubricant and assist the aggregate to attain its closest packing. The binder is present as a thin film surrounding each particle of aggregate, and additional binder causes an increase in thickness of these films, thereby reducing the frictional resistance to compaction. Hence up to a certain point, there is diminution in the total volume as the binder content is increased. After this point has been reached, further additions of binder, by increasing the film thickness, will cause an increase in volume."

With the realization that the cohesion component of the shear strength of bituminous mixtures is affected by asphalt content, the relation between the optimum asphalt content and density for a given set of materials to the maximum shear strength of the mixture becomes of critical importance. Neppe (1) and Endersby (26) point out that the presence of the binder complicates the determination of the stability of the mixture. Increased quantities of bitumen increase the cohesion of the mixture but simultaneously it acts as a lubricant which tends to reduce stability. Mixture design methods often indicate that maximum stability of a mixture is achieved with the combination of aggregate and bitumen that produces maximum density. This observation, in general, does not hold true for triaxial test evaluations of shear strength, where it is usually observed that maximum stability occurs for asphalt contents less than those required for maximum density. Goetz and Chen (46), for example, note that their test results show, "...maximum stability at any confining pressure does not occur at the asphalt content which yields maximum density. The asphalt content at maximum stability is less than the asphalt content at maximum density."

Analysis of Strength Test Data

A graphical representation of the results of an open-system triaxial test series is normal procedure for the analysis of the shear strength of bituminous mixtures. Utilizing the Mohr theory, the shear strength of the mixture is presented as a cohesion and a friction component. An extensive search of the literature on triaxial testing of bituminous mixtures has failed to reveal any detailed discussion of the effects of drainage conditions during the test on shear strength. If drainage conditions during the test affect observed shear strength values, then the triaxial test data available must be considered as c and ϕ values apropos to the particular test conditions under which they were determined. This consideration makes it impossible to compare results from different agencies and furthermore, it limits the determination of the fundamental shear resistance characteristics of bituminous mixtures.

There is direct and implied test evidence to indicate that there is a form of drainage from a bituminous-mixture specimen under load. The drainage most generally occurs from the air phase, but the asphalt also may be forced from the specimen under load. Campbell (47) investigated the one-dimensional compressibility of bituminous mixtures. His results indicate that under an applied load there is relatively sudden compression of the mixture followed by a lesser and slower reduction in the height of the sample with time. The former compression was thought to be due to compression and drainage of air from the sample while the latter was considered to be due to movement of asphalt. Campbell noted further: "It was observed at the completion of each test

that asphalt had actually been expelled from the samples and forced into the faces of the porous stones." McCarty (48) points out that the application of the Coulomb equation to bituminous mixtures must be revised in light of the Hvorslev modifications that have been accepted by soils engineers (see "Triaxial Testing of Soils"). Numerous other authors refer to changes in density of specimens during testing which must be considered as an implication of drainage either from the specimen or into it. Nijboer (49), for example reports: "If it is realized that in the triaxial test all principal stresses on the test piece increase, involving a reduction in volume of the specimen, it must be inferred that the void content of the aggregate will be reduced during testing."

If the hypothesis of the possibility of drainage of some type from a sample of a bituminous mixture subjected to triaxial testing is accepted, it would seem desirable to establish the effects of test drainage on observed shear strength and stability values.

Using the existing test procedures (no particular regard to drainage) and the Mohr circle analysis, the phenomena of the curved envelope of failure has been observed frequently. The occurrence of the curved envelope has caused extensive concern among paving engineers and no explanation of the cause is accepted universally. McLeod (50) candidly states: "Not enough appears to be known about the triaxial testing of bituminous mixtures to establish why straight line Mohr envelopes are obtained for some and curved Mohr envelopes for others." The wide divergence of authoritative opinion on the cause of curved envelopes of failure is well illustrated by the following quotation from McLeod (51).

> "Professor Vallerga states that 'the curvature of Mohr's envelope of failure and the apparent intercept on the ordinate for these materials are merely effects produced by particle size relative to specimen size, size of specimen, kind of loading, limitations of test equipment and instrumentation, and finally improper interpretation of test data'. Mr. Endersby believes that 'the curvature represents the structural effect pure and simple, and the extent of it depends upon the dimensions of your sample'. According to Mr. Carpenter, 'we get curvature when there is a change in density of the specimen during test', while Dr. Mack states, 'the hardening effect can be also demonstrated by a curved envelope to the stress circles.' ...It is quite apparent, therefore, that considerable difference of opinion exists concerning the cause of the curvature of the Mohr envelope."

Nijboer (49) notes, "With respect to Mohr's diagrams, it should be stated that the mixtures with a low bitumen content show the normal picture, but for mixtures containing more bitumen and having a void content below 2-3% by volume the enveloping curve of the circles is no longer a straight line, but runs concavely towards the σ-axis (horizontal axis)."

McCarty (48) believes, "The deviation of the Mohr envelope from this straight line relation may be interpreted as the effect of a change in cohesion, due to compaction of the specimen, or as a consequence of changes in structural properties of the material due to internal rearrangement of aggregate particles in the mix, etc., which is a necessary accompaniment of the changing consolidation." Eatman (52) reiterates McCarty's belief.

Considering the weight of evidence that bituminous mixture specimens change in density during a triaxial test and recalling the explanation of the curved envelope of failure that has been accepted for partially saturated soils, it is likely that a similar explanation is

reasonable for bituminous mixtures. If this be the case, the need for establishing the effect of test drainage conditions on observed shear strength and accurate measurements of the change in volume of the specimen under all loads applied during the triaxial test becomes more critical. It is this line of thought that led to the research reported herein.

Relationship of Air Voids to Mixture Design

Any study that contemplates measuring the change in volume of a bituminous mixture specimen under load must consider the question of the voids in the mixture. If it is assumed that the aggregate particles and the bitumen are incompressible, then a decrease in sample volume may be accomplished only by a decrease in air-void volume.

The majority of established mixture design procedures presently in use specify or suggest limits to the amount of voids in the compacted specimen. The concepts reflected in the establishment of void limitations are expressed well in Highway Research Board Bulletin No. 105 (53):

> "In designing bituminous paving mixtures of the dense-graded type, it is desirable to have the voids in compressed specimens above a minimum percentage and below a maximum percentage. The minimum percentage of voids should be such as to allow for increased densification by traffic loads and for expansion of the aggregate caused by temperature increase to avoid exuding bitumen on the surface and consequent reduction in skid resistance and stability. The maximum percentage of voids should be set at such a point as to insure sufficient density together with other desirable properties, such as stability and tensile strength."

It should be noted that the above statement recognizes two important facets of the present study: (a) density may increase and therefore there is a decrease in the volume occupied by a given quantity of material and (b) a decrease in stability (after reaching a certain

point) accompanies the increased densification.

Though the decrease in voids due to traffic compaction after a bituminous pavement is in place is universally recognized there appears to be little reference in the literature of the possibility of void changes during testing. Endersby (54) and Nijboer (49) recognize that volume changes take place but their discussions fail to present any significant data on the magnitude of these changes. Endersby (26) notes that for the mixes tested there was a decrease in density upon initial deformation and an increase in density thereafter. Lee and Markwick (31) observed: "Compacted bituminous materials increase slightly in volume when deformed." These authorities compare this action to that of granular materials known as 'dilatancy' and indicate that dilatancy is roughly proportional to deformation and is greater for mixes with low percentages of bitumen. Endersby (54) suggests the possibility of a critical void ratio for bituminous mixtures: "That is to say, a given mix, when compacted below a certain void content, will expand continuously during deformation. If the voids are above this point, the material will compact during deformation."

The effect of voids on stability is also the subject of much discussion. Hveem and Vallerga (45) state: "...there is evidence to show that when carried too far, reducing air voids in an asphalt pavement may result in other undesirable developments - loss in stability for example." In an earlier paper, Stanton and Hveem (25) present the following: "It is difficult to find evidence to prove that low void volume is the sole or even an essential measure of quality." However, Endersby (54) found: "...the change in stability resulting from

decreasing voids in a given mix, varies inversely somewhat as a straight line with voids." Neppe (1) discusses the effects of low void contents as follows: "It should however be realized that in a compacted mineral aggregate there is a continuity between individual voids by the interlocking of contiguous particles. If, however, the voids in the compacted mineral aggregate are overfilled, the surfaces of adjacent aggregate particles will be kept from making intimate contact and the stability of the mixture must inevitably be greatly reduced by the lubricating action of the binder."

Nijboer (49) draws two important conclusions regarding the stability of mixtures with very low percentages of voids. He indicates that for mixes with less than two percent voids a part of the externally applied pressure is carried by liquid phase of the mixture and that a very rapid decrease in resistance occurs as void contents are reduced to two-three percent. The first of these conclusions introduces the concept of pore pressures and effective stresses to the stress system of a bituminous mixture and implies that, in a fashion similar to soil, there would be a decrease in neutral stress and an increase in effective stress if time were provided for drainage. The reason for the limitation of this concept to mixtures of less than two percent voids is not apparent. The second conclusion is a direct outgrowth of the first for as effective stresses are decreased so would the shearing resistance of the mixture be reduced.

All of the preceding comments on the effects of voids in a mixture deal with their affect on stability which is the primary concern of this investigation. It should not be overlooked, however, that voids

influence other characteristics of mixes; in particular, the durability of the mixture. McLeod (55) draws attention to this aspect of voids considerations as follows: "The achievement of pavement durability requires careful consideration of the void properties of compressible bituminous mixtures." With high void contents, the mixture is subject to easy penetration of air and water and the deterioration of the mixture is hastened.

Influence of Specific Gravity Values

The evaluation of the percentage voids in a mixture is complicated by the weight-volume relationships that exist between aggregate and bitumen. It is common practice to design mixtures on the basis of weight and to compute the voids present in a compacted mixture by the difference in the volumes occupied by the solid and liquid ingredients and the volume of the compacted mixture. The adequacy of solutions established by this scheme of computations is directly related to the specific gravities of the ingredients that are used in the calculations. Ricketts, et al (56) summarizes the status of the various aggregate specific gravities:

> "The three conventional specific gravities have been used more or less interchangeably for bituminous aggregates, some agencies using bulk, some apparent, and some saturated surface-dry specific gravity; voids criteria of course must be consonant with the specific gravity used. When using aggregate of low absorption (less than 0.5%), the induced error is not too great; but when the more permeable aggregates are used the conventional specific gravities can provide unsatisfactory results."

The unsatisfactory results that are possible depend upon the absorption of bitumen by the aggregate. If the aggregate absorbs no bitumen, the

bulk specific gravity would yield correct values for the volume of aggregate; if the bitumen absorption is the same as water absorption the apparent specific gravity would yield correct results. However, in general, the bitumen absorbed by an aggregate is less than the amount of water the same aggregate would absorb but is some tangible value greater than zero. Therefore, voids computations using either specific gravity are incorrect. An aggregate specific gravity that is computed using a volume that provides an allowance for the bitumen absorbed would be the most correct value to use for void computations. The Bulk-impregnated Specific Gravity satisfies the need for a specific gravity measured as indicated (57).

A second approach to the problem of evaluating the effect of absorbed bitumen in specific gravity was developed by Rice (58)(59). In this procedure, a Maximum Theoretical Density of an uncompacted mixture of aggregate and bitumen is measured. This value may be used to compute an effective specific gravity of the aggregate which includes the effect of the absorbed bitumen. Because of the relative simplicity of the method many investigators prefer to work with this procedure.

Still other methods of accounting for the absorption of bitumen by aggregate are available (60)(61). Recent results of Hode-Keyser (62) indicate that the Rice method values are the most consistent and the most correct for a wide variety of aggregates of all of the proposed methods of evaluating the effect of absorbed bitumen on the specific gravity of aggregates.

All of the procedures indicated above provide means for computing a value for the percent absorption of bitumen by aggregate. McLeod (55)

recommends that the values for voids mineral aggregate and void content be computed on the basis of the ASTM Bulk Specific Gravity (63) and an allowance for the bitumen absorbed by the aggregate. This procedure defines the volume occupied by the aggregate components and reduces the volume occupied by the bitumen by the amount of bitumen absorbed by the aggregate. The net result is that the percent voids computed represent the space in a mixture that is free of solids or liquids as accurately as possible within the present knowledge of the subject. In addition, by this method of analysis, the asphalt content would be based on the amount actually available to fill the voids between mineral aggregate particles.

On the basis of the literature reviewed, it would appear that the objectives of this research constitute a necessary next step in the development of the technical knowledge of the performance of bituminous mixtures under triaxial compression loading.

MATERIALS AND PROCEDURES

This section of the report describes all details relating to the tests performed including a description of the materials used, characterization test results, the results of a study on sample fabrication procedures, the equipment developed for the investigation, and the procedures followed in the triaxial compression testing of all samples.

Materials

The bituminous concrete mixture utilized for this study was chosen to be typical of such mixtures in use at the present time by various agencies. It was not intended that the mixture satisfy the specifications of any one organization. However, the mixture does satisfy, in general, the requirements of the State Highway Department of Indiana for Hot Asphaltic Concrete Surface, Type A (64).

The composition of the aggregate blend used is shown in Table 1, Table 2, and Figure 1. In Figure 1 the gradation of two other mixtures are shown for comparison purposes.

The coarse aggregate (percent retained on No. 4 sieve) utilized in this study was a crushed and washed limestone from central Indiana (laboratory No. 67-2S). The limestone source is located in the St. Genevive formation and is of Mississippian age. The limestone is characterized by a low porosity and a fine-grained texture. The values of specific gravity established by ASTM Method C-127 (66) are as follows:

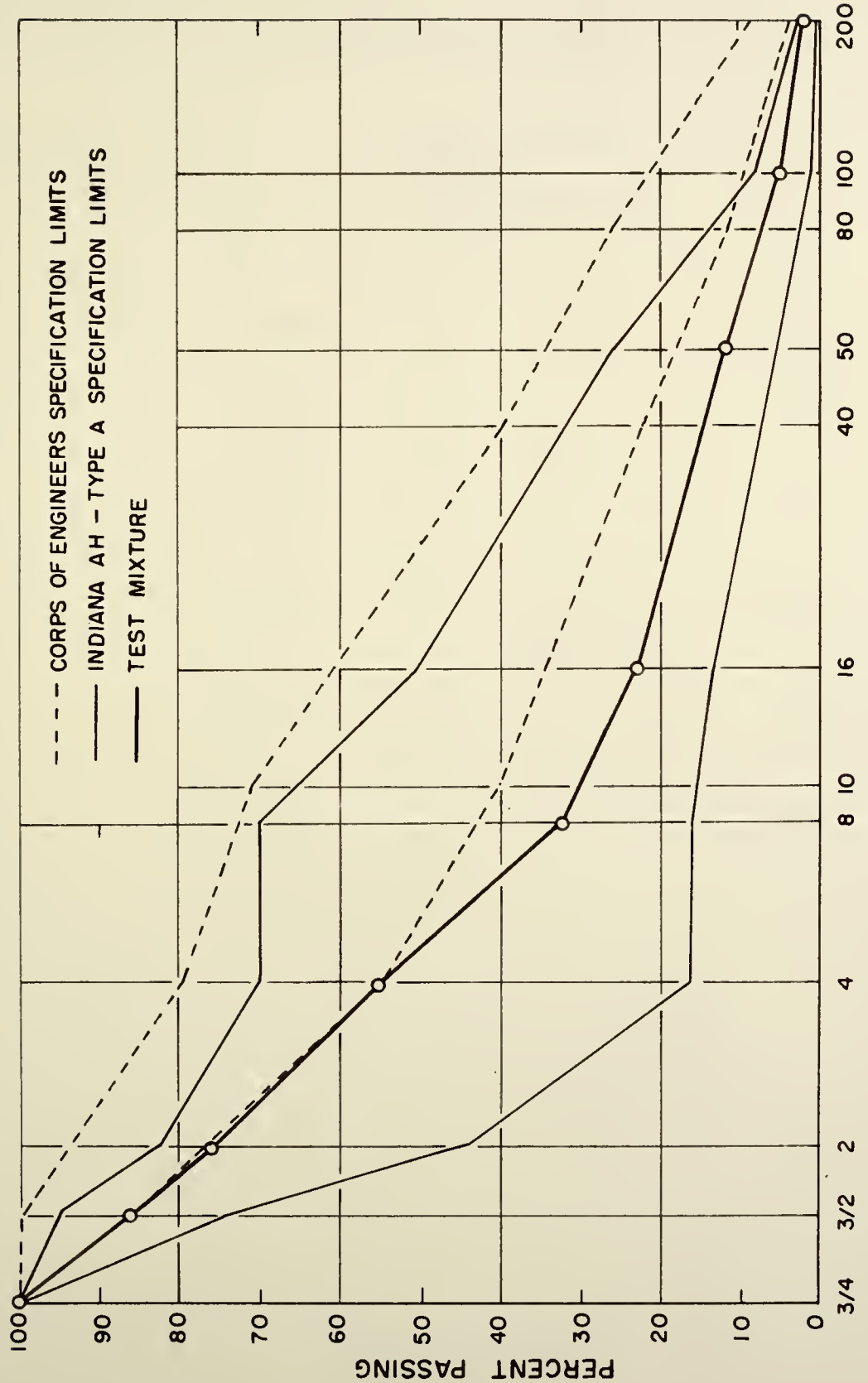


FIGURE 1. AGGREGATE GRADATION FOR TEST MIXTURE

TABLE 1

Aggregate Gradation Data

Sieve No.	Percent Passing		
	Indiana Type A (64)	Corps of Engineers 3/4 Maximum Surface Course (39)	As Used
3/4	100	100	100
1/2	75-95	86-100	86.0
3/8	45-83		76.0
4	16-70	55-80	55.0
8	16-70		32.0
10		40-66	
16	13-50		22.5
40		22-40	
50	5-25		12.0
80		12-26	
100	1-8		5.0
200	0-3	4-9	2.0

TABLE 2

Aggregate Gradation of Test Mixture

Passing Sieve No.	Retained Sieve No.	As Used, %
3/4	1/2	14.0
1/2	3/8	10.0
3/8	4	21.0
4	8	23.0
8	16	9.5
16	50	10.5
50	100	7.0
100	200	3.0
200	-	2.0

Bulk Specific Gravity	2.660
Apparent Specific Gravity	2.700
% water absorption	0.63%

The fine aggregate (percent passing the No. 4 sieve and retained on the No. 200 sieve) was obtained from two sources. The portion passing the No. 100 sieve was a very uniform dune sand obtained near Monon, Indiana. The remainder of the fine aggregate was obtained from a local river terrace deposit (laboratory sample No. 79-1). The specific gravity values obtained by using ASTM method C-128 (63) were:

	No. 79-1	Dune Sand
Bulk Specific Gravity	2.574	2.595
Apparent Specific Gravity	2.715	2.656
% water absorption	2.01	0.88

Portland cement was used for the filler (percent passing the No. 200 sieve). The material used was a Type I cement with a specific gravity of 3.15 and satisfied all requirements of ASTM designation C-150 (65). Portland cement was chosen for the filler to insure a supply of uniform material meeting the desired gradation requirements and for its ease in handling in the laboratory.

As the question of voids in the compacted mixture was critical to the study, it was desirable to obtain the percentage of bitumen absorbed by the aggregate blend. This problem was approached by two different methods: (a) Bulk-impregnated Specific Gravity (56) of the aggregates and (b) computed from the Maximum Theoretical Density (59) of the aggregate blend. Using the procedures outlined in the references indicated for the former process, the following values were obtained:

	Bulk-impregnated Sp. Gr.	% bitumen absorbed
Coarse Aggregate	2.671	0.15
Fine Aggregate	2.642	1.01

The filler material was assumed to absorb no bitumen. Using the above data, an average bulk-impregnated specific gravity of the blend of 2.664 was computed and a value of bitumen absorbed for the aggregate blend was found to be 0.61%.

With the Maximum Theoretical Density (Rice) procedure, an average effective specific gravity of the aggregate blend was computed to be 2.672 and the percent bitumen absorbed was found to be 0.72%.

It was considered that the values established by the Maximum Theoretical Density procedure were more apropos to this study as they reflect the measured characteristics of the blend whereas the Bulk-impregnated Specific Gravity procedure established these values by computation from determinations on the aggregate fractions. With the excellent correlation of the results of the two procedures, the choice of test values for use in design is relatively unimportant.

An 85-100 penetration grade asphalt cement was provided by the Texas Company from the Port Neches refinery for use in this study. Test results on the asphalt are presented in Table 3.

The aggregate blend chosen and the range of asphalt contents considered for the testing program were evaluated for general suitability as an asphalt paving mixture by means of the Marshall (40) and Stabilometer test procedures (41). The results of these characterization tests are shown in Table 4.

TABLE 3

Results of Tests on Asphalt Cement

Penetration, 100 g., 5 sec., 77°F	93
Penetration, 100 g., 5 sec., 32°F	21
Softening Point, Ring and Ball, °F	116
Ductility, 77°F, cm.	200+
Flash Point, Cleveland Open Cup, °F	590
Solubility in CCl_4, %	99.86
Spot Test	Negative
Specific Gravity, 77/77 °F	1.033

TABLE 4

Results of Characterization Tests

Marshall Method Results

Asphalt Content % by weight of mix	Stability lbs.	Flow 1/100 in.	Unit Weight lb/cu.ft.	Voids %	Voids Filled %
3.75	1105	11.5	145.7	6.2	58.0
4.25	1165	13.3	147.3	4.6	69.0
4.85	1125	13.8	149.1	2.6	82.0
5.19	1232	14.3	149.5	1.8	87.5
5.74	1050	15.9	149.1	1.1	92.5

Stabilometer Method Results

CKE, fine aggregate	2.2
Oil Equivalent, coarse aggregate	3.3
Surface Area, sq.ft/lb.	18.5
Optimum Asphalt Content, % by weight of aggregate	5.2

Asphalt Content %					
by weight of mix	by weight of aggregate	Stabilometer Value	Unit Weight lb/cu.ft.	Voids %	Voids Filled %
4.06	4.23	45	150.4	4.7	67.5
4.27	4.46	43	151.4	3.9	72.7
4.77	5.01	37	152.7	2.4	83.0
5.32	5.63	31	152.3	1.7	88.5

Batching and Mixing

The aggregates used were batched by component fractions according to the blend formula. Batching was accomplished with cold dried aggregates using a torsion balance sensitive to 0.1 g. The blended aggregates and asphalt for a batch were heated separately to 300°F ± 10° in a Peerless gas oven. At the same time the mix ingredients were being heated, the mixing bowl, paddle and miscellaneous spoons and spatulas were heated. The combining of the aggregate and asphalt for a batch was accomplished using a Toledo scale (10 Kg capacity, 1 g direct reading). A tare weight of the mixing bowl was recorded, the hot aggregate was added to the bowl, and the combined weight noted. The difference in this weight and the tare weight of the bowl established the actual weight of hot aggregate used in the batch and was the weight used in all asphalt content computations. The heated asphalt cement was added with the mixing bowl and aggregate on the Toledo scale. The direct reading fan of the scale permitted control of the amount of asphalt added to the nearest one gram. A check on the weight of asphalt cement used was made by noting the difference in weight of the pan of asphalt before and after adding the material to the aggregate. A typical mix record is shown in Appendix B.

With the desired quantities of hot materials in the bowl, mixing was accomplished using a Hobart electric mixer (Model N-50) modified with a special mixing paddle and scraper. The mixing was continued for two minutes.

After mixing, the batch was placed in a shallow, flat-bottomed metal pan which had been heated to 140°F. Care was exercised to obtain, as

nearly as possible, all material from the mixing bowl and mixer paddles. As two batches were necessary to provide the material necessary for the fabrication of one specimen, the above mixing procedure was repeated while the first batch was stored in a 140°F oven (Electric Hotpack Model 14-12) to prevent excessive cooling. The two batches were combined and hand-mixed in an effort to obtain a single mass of uniform material.

The mixed material was cured in the shallow pan for a period of 18-20 hours in a forced-draft oven at 140°F prior to compaction.

Compaction

Three methods of compaction were available for the formation of specimens of the size desired for this study. Dynamic compaction would have required an intensive study of the relationship between height of fall, weight of tamper, number of blows and number of layers in order to arrive at a satisfactory procedure to fabricate an eight inch high specimen. The problem of developing a reasonably uniform density throughout the height of the specimen was a major objection to static compaction. The third method, kneading compaction, seemed to offer the best possibilities for providing a desirable specimen. In addition, this method of compaction was consonant with the present trend of thought regarding the formation of a specimen most nearly like the pavement prototype.

The principles of kneading compaction have been discussed frequently as has been the operation of the kneading compactor (66)(44)(38). In general, the compaction process is accomplished as follows: Material is fed into the mold which is mounted on a rotating table. The compaction foot, a pie-shaped segment with an area approximately one-fourth of that of the mold, is activated by a combination mechanical and

hydraulic-pneumatic control system. The foot moves downward against the sample until a pre-set load is built up. The load is maintained for a short interval of time and then released. A load cycle requires two seconds to complete. Means are available for adjusting the foot pressure over a wide range of pressures. Material may be placed in the mold in layers or by a continuous flow process during compaction.

The kneading compactor model available for use is shown in Figure 2. It was constructed by the August Manufacturing Company of Oakland, California. A special mold, Figure 3, suitable for the formation of specimens eight inches high by four inches in diameter was constructed in the shops of the School of Civil Engineering.

The general scheme of compaction considered for this study consisted of placing a portion of the sample mixture into the mold, rodding with a three-quarter inch diameter bullet-nose rod, and applying a given number of tamps after which the remainder of the mix was fed into the mold while compaction was continuing. A final number of compacting tamps on the surface of the sample completed the sample fabrication. The problem was to establish the most desirable combination of tamps, foot pressures, etc. for the fabrication of specimens from the mixture to be used for the study.

The criteria for a suitable specimen was considered to be uniform density and asphalt content throughout the height of the sample. Uniformity of these two measures implies a uniform distribution of voids throughout the specimen. Void uniformity was considered desirable in light of the purposes of the study. To evaluate the uniformity of asphalt content and density, each specimen made for purposes of developing

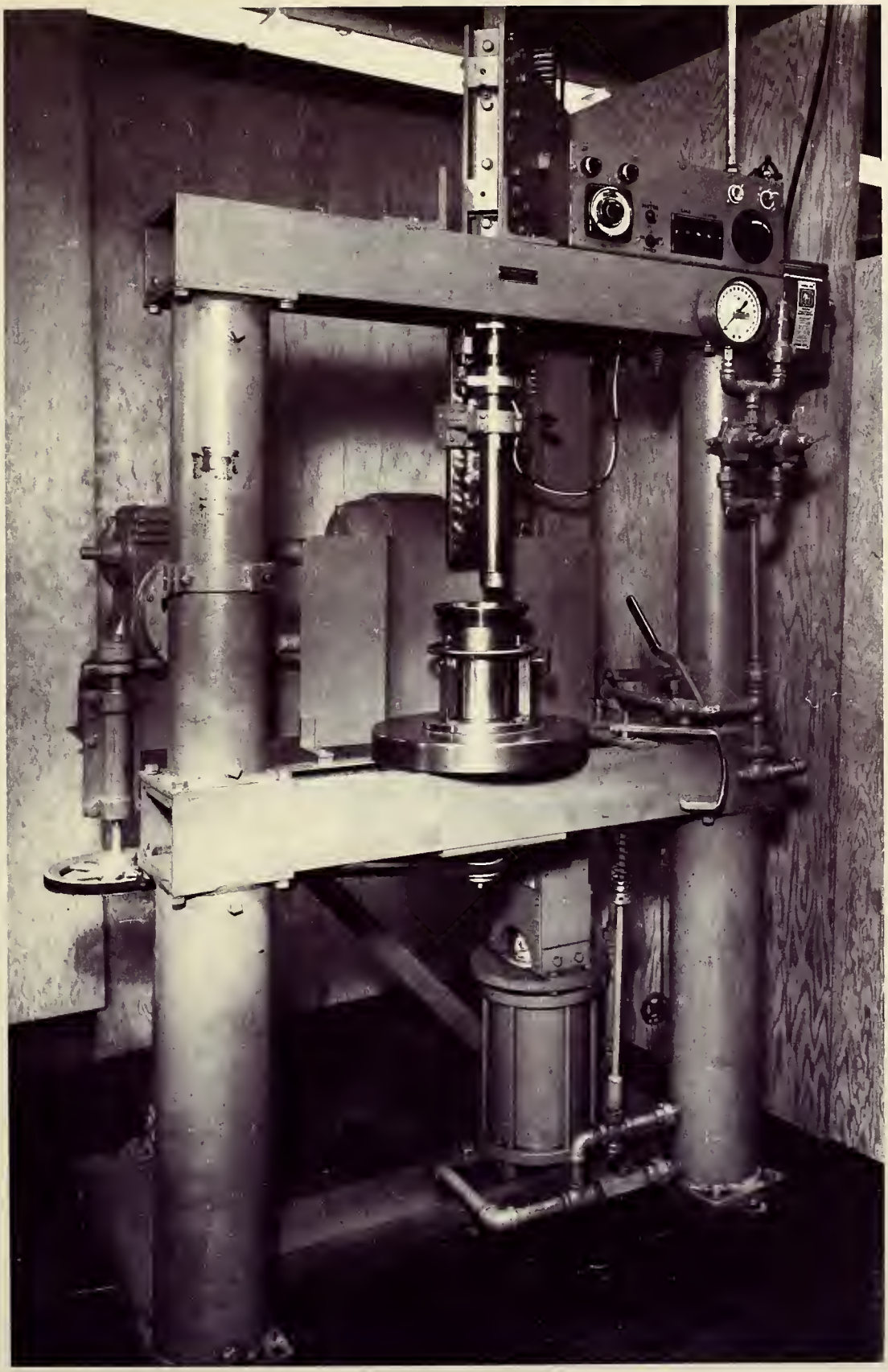


FIGURE 2. KNEADING COMPACTOR

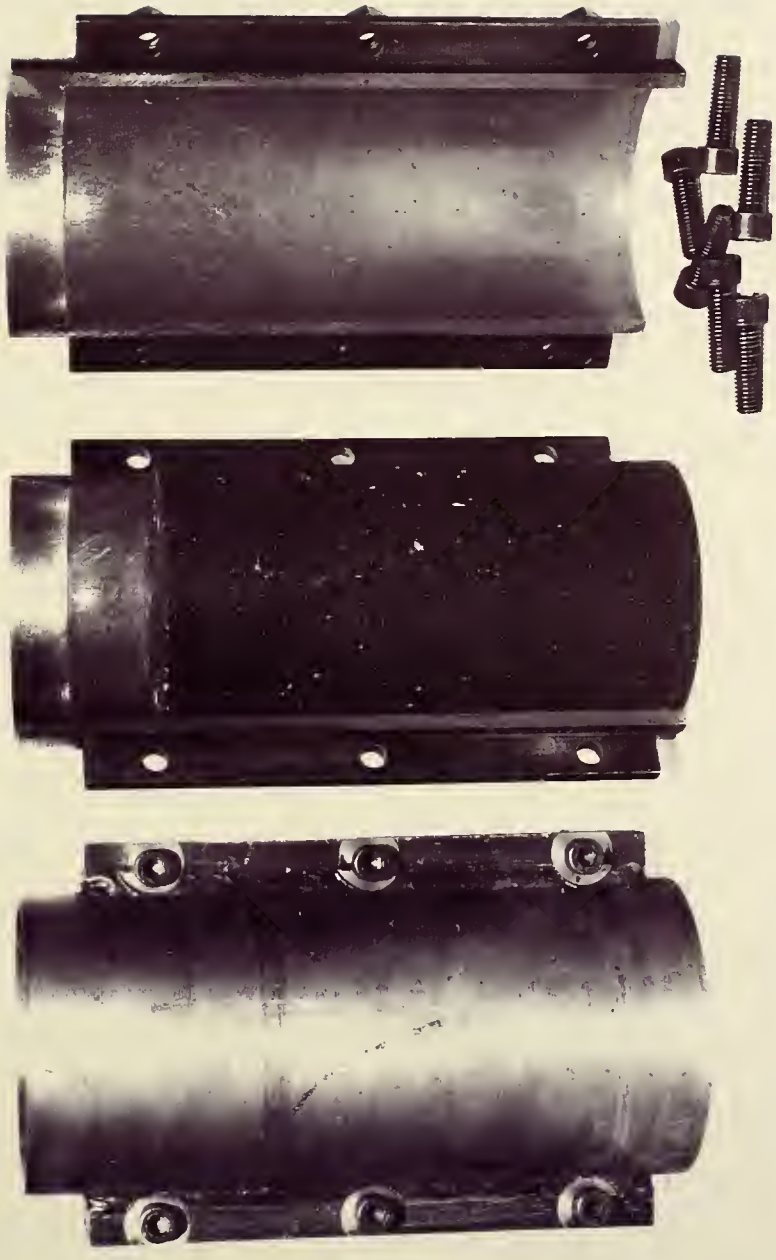


FIGURE 3. MOLD FOR TRIAXIAL TEST SPECIMENS

a compaction procedure was cut into three sections perpendicular to the axis of the sample. Each section was measured, weighed, and the volume computed by displacement in water using the principle of Archimedes and by computation using the specimen dimensions. The sections were then treated for the extraction of asphalt using ASTM Method D-1097 (67).

Absolute uniformity of density or asphalt content was considered to be unattainable from a practical standpoint. It was decided, therefore, to strive for a procedure which yielded density values of the sections within one percent of the average of the three sections and an asphalt content of the sections within three-tenths of one percent of the average asphalt content of the three sections as established from extraction test results.

Numerous trial compaction procedures were investigated. The procedure finally settled upon was as follows:

1. The mixture was removed from the 140°F curing oven after 18-20 hours of curing and was remixed by hand.
2. The mixture was split into two approximately equal parts and each part brought to a temperature of 230°F $\pm$ 10°.
3. One-quarter of the mixture was introduced into the preheated mold which was placed on the rotating table of the kneading compactor.
4. To reduce the tendency for surface voids to develop in the lower portion of the sample, the first one-quarter of the mixture was rodded with a 3/4 inch bullet-nose rod. Twenty tamps were applied to the center of the mix and twenty around the periphery of the mix.

5. The material in the mold was subjected to 15 tamps of the tamping foot under the desired and pre-set foot pressure.
6. The remainder of the sample was introduced into the mold while the compactor was in operation. One-third of this portion (one-quarter of the total sample) was added gradually over each of three two-minute periods. All material was stored in the heating oven at a temperature of 230°F ± 10° until ready for compaction. This procedure permitted the application of 180 tamps on three-fourths of the original sample while this quantity was being added to the mold.
7. Two minutes of tamping (60 tamps) was applied to the surface of the entire sample.
8. The mold was removed from the kneading compactor and the sample subjected to a double plunger static load of 12600 pounds as a leveling load. This load was applied at a rate of 0.025 inches of deformation per minute and was released immediately upon reaching the load specified. The loading was applied by the use of a Riehle Testing Machine, 50,000 pound capacity, and having a variable speed drive.
9. The mold was allowed to cool in air for approximately 30 minutes, at which time the mold was disassembled and the specimen removed.
10. Each specimen was marked for identification. The bulk density of each was determined from the dimensions of the specimen for purposes of establishing the uniformity of the specimens.

11. All specimens were stored in a constant temperature atmosphere of 77°F until time for testing.

Tables presented in Appendix C show typical density and extraction asphalt content results for specimens made by the foregoing procedure at asphalt contents by weight of mixture of 2.8, 4.0, and 5.0 percent and at 150, 250, and 400 psi compaction foot pressures. It may be noted in all cases that the uniformity of density was well within the limit sought. In the majority of cases, the density of the sections varied from the average by less than one-half of one percent. The uniformity of asphalt content that was desired was attained in all specimens.

Controlled Temperature Room

In order to eliminate the variable of temperature and its effects upon the tests performed in this study, a controlled-temperature atmosphere was considered necessary. A commercially available unit marketed by The Electric Hotpack Company, Inc., Philadelphia, Pennsylvania, under the name Controlled Environmental Room, Model 8316, was found to satisfy all requirements of the project and was obtained for use.

Figure 4 is a photograph of the exterior of the controlled-temperature room while Figure 5 shows the interior of the room with all testing equipment in place. Figure 6 shows a close-up view of the control panel and recording unit on the exterior of the room.

The room obtained was 6 x 8 x 7 feet and was assembled in the laboratory from four prefabricated sections. All portions of the room in contact with the surrounding atmosphere contained three inches of precompressed glass wool as insulation. Temperature control was provided

FIGURE 4. CONTROLLED TEMPERATURE ROOM–
EXTERIOR VIEW

FIGURE 5. CONTROLLED TEMPERATURE ROOM—
INTERIOR VIEW

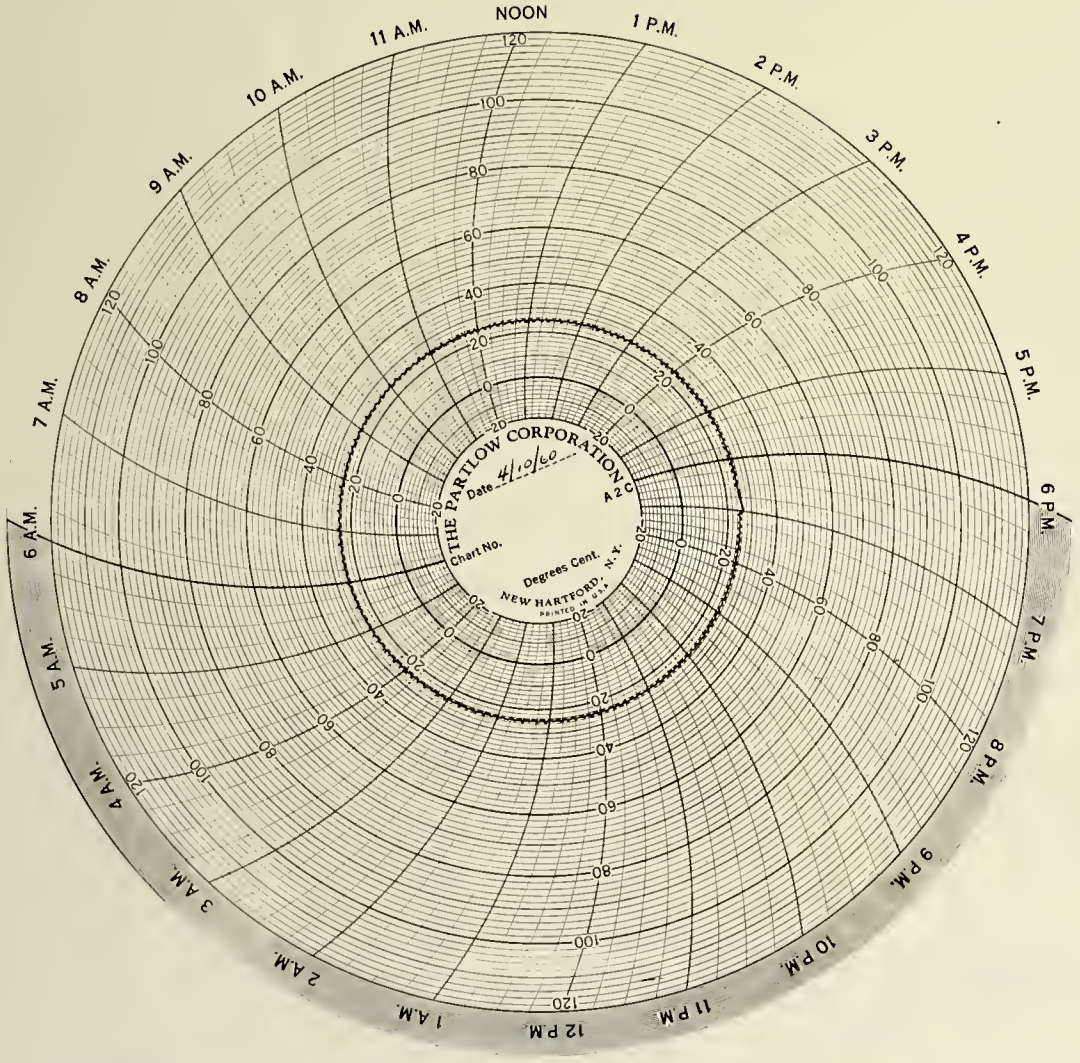


FIGURE 6. CONTROL PANEL, RECORDING UNIT AND TYPICAL TEMPERATURE RECORD FOR CONTROLLED TEMPERATURE ROOM

through Partlow thermostatic controls operating heating coils and a refrigeration unit. Humidity control was an integral part of the room but was not utilized for this study. The room was rated electrically as operating on 230 volt, single-phase AC current and requiring approximately 4800 watts of power.

The recording unit provided a continuous record of the room's temperature through a 24 hour period. Tests conducted during the early period of operation of the room indicated that the recorded trace was an accurate measure of the dry bulb temperature of the room at the sensing element. A series of accurate thermometers were placed at points of interest throughout the room and observed carefully for long periods to note the variation of temperature within the room. Average air temperatures at the triaxial cell and at the volume measuring device were observed to be 0.3°C above the temperature at the sensing element. Temperature readings taken every minute on all thermometers indicated a cycling of temperature from approximately 1°C above the average to approximately 1°C below the average with the cycling occurring approximately once every ten minutes. A typical set of observations are presented in Appendix D.

Observations on the water temperature within the elements of the volume measuring device showed a maximum variation of $\pm$ 0.1°C from the temperature at the sensing element. In summary, all temperature observations indicated that the average temperature at any point within the room was within the desired maximum temperature range of $\pm$ 0.5°C. A typical temperature record, as made by the Partlow recorder, is shown in Figure 6.

Volume Measuring Device

The basic idea which resulted in this study was a belief that bituminous-mixture specimens change volume upon the application of various test loadings. It was necessary, therefore, to develop a method of measuring the specimen volume or change in volume at various stages during the performance of a triaxial test.

Several schemes were considered to provide a procedure for the measurement of the volumes of the specimens under test conditions. The most promising method was one adopted from a procedure used by the Soil Mechanics Laboratory of the University of London and reported by Bishop and Henkel (23). Figure 7 is a photograph of the device used in this study while Figure 8 is a diagrammatic sketch of the volume measuring device that also shows its relationship to the triaxial cell and the pressure source. The key following these figures identifies the principal parts of the apparatus (Table 5).

The basic principle upon which the volume measuring device was operated is simple. Any change in the volume of the sample was reflected by a change in the volume of the liquid in the triaxial cell. The magnitude of this volume change was measured by a change in the level of the mercury column in the measuring tube. A differential head of mercury between the measuring tube and the mercury reservoir tube was prevented by mounting the latter tube on a calibrated spring that changed length as the amount of mercury in the reservoir tube changed.

The volume measuring device was operated as follows: The triaxial cell was filled with the confining liquid, water in this case, with care taken to remove all air bubbles from the cell. At this point, the water

FIGURE 7. VOLUME MEASURING DEVICE

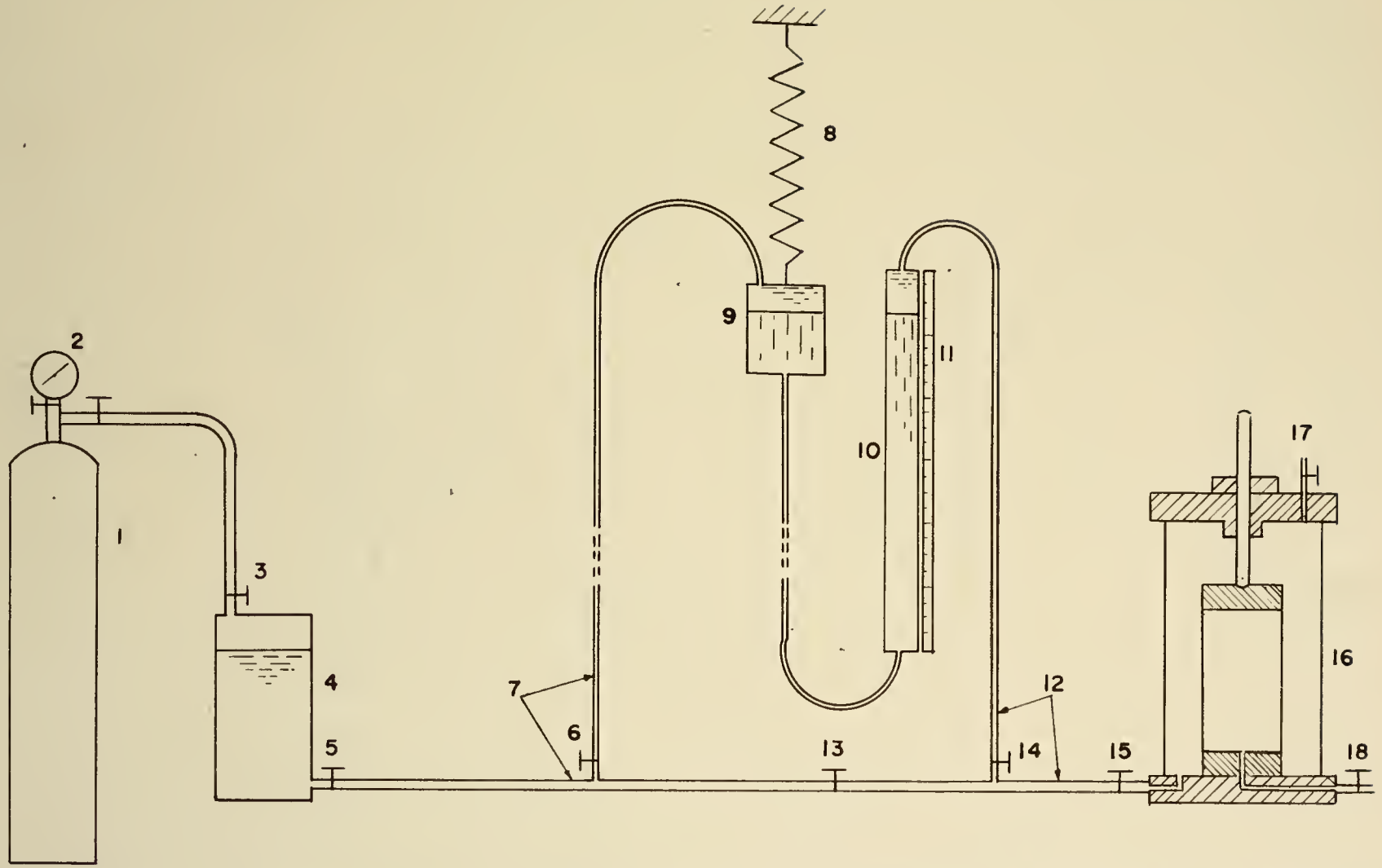


FIGURE 8. DIAGRAMMATIC SKETCH OF VOLUME MEASURING DEVICE

TABLE 5

Key to Details of Figure 8

1. Nitrogen tank
2. Pressure regulator
3. Quick release pressure coupling
4. Water reservoir tank
5. Water reservoir tank outlet valve
6. Mercury reservoir control valve
7. Polyethylene plastic tubing (1/4 inch O.D.)
8. Calibrated spring
9. Plastic mercury reservoir tube
10. Plastic measuring tube
11. Metric scale
12. Polyethylene plastic tubing (1/4 inch O.D.)
13. Bypass valve
14. Measuring tube control valve
15. Triaxial cell water inlet valve
16. Triaxial cell
17. Triaxial cell air escape valve
18. Sample drainage valve

in the cell was continuous to the water in the reservoir while the flexible tubing and the measuring tubes were completely filled with water and mercury. Valves were adjusted so that a pressure applied to the water reservoir was transmitted through the flexible tubing to the liquid in the cell. Upon the application of pressure to the cell liquid (the confining pressure of the test), a change in the level of the mercury in the measuring tube took place. This change reflected the increase in the volume of the cell liquid that resulted from the lateral pressure forcing the impervious membrane into the surface voids of the specimen. The change in volume of the specimen with time under any load application to the specimen caused a change in the level of the mercury in the measuring tube. For example, if the specimen compacted or decreased in volume under load, water entered the cell from the flexible tubing and the mercury level of the measuring tube moved upwards an amount equal to an equivalent volume. Simultaneously the mercury level in the reservoir tube dropped. The decrease in weight of the reservoir tube caused the calibrated spring to shorten until a uniform level of mercury existed between the two mercury tubes.

The metric scale located adjacent to the measurement tube permitted a reading of the change in level of the mercury to the nearest one millimeter which corresponded to a volume change of 0.497 cubic centimeters. A volume change of this magnitude is equivalent to a change in volume of the specimen of approximately 0.03 percent. It should be noted that any observed change in mercury level must be corrected for any variation in temperature, for the change in volume of the cell under the applied confining pressure, and for the volume displaced by the movement of the piston.

Temperature changes were minimized by the use of a controlled temperature atmosphere in which to perform all tests. A maximum temperature variation of one-half degree Centigrade was established for the test location. With these temperature conditions the maximum possible variation of the volume of the cell liquid and the volume of the cell has been computed to be $\pm$ 0.255 cubic centimeters or a variation in mercury level of approximately one-half millimeter.

The change in the volume of the cell and the volume measuring device under the applied confining pressure was established experimentally by calibration tests. A steel dummy specimen the same size as a bituminous specimen was used to simulate it and the cell was assembled as for a routine test. The changes in volume of the apparatus under the various confining pressures to be used were observed and recorded as calibration factors for the assembly. The change in volume of the assembly under a confining pressure occurs almost instantaneously upon the application of the pressure. Little change in volume occurred with time under prolonged application of the confining pressure.

The entrance of the piston under load into the triaxial cell caused a continuing displacement of the cell liquid that resulted in a decrease in the level of the mercury in the measuring tube. Computations based on the diameter of the piston and the deformation of the specimen permitted the evaluation of a correction factor which was applied to apparent volume changes.

Observations of temperature variation within the controlled-temperature atmosphere, test measurements of the change in volume of the apparatus under the various confining pressures used and with time, and

the magnitude of the volume changes due to piston displacement are presented in Appendix D.

Triaxial Testing Equipment

Figure 5 and Figure 9 are general views of the triaxial testing equipment used in this study. The former figure shows a front view of the testing frame in position within the controlled-temperature room while the latter figure is a close-up of the loading crossbar of the frame with the triaxial cell in position. Figure 10 shows a section of the triaxial cell and should be used in conjunction with Table 6 which serves as a key to the major elements of the cell.

The loading frame was constructed in the shops of the School of Civil Engineering. In general, it is similar to a consolidation frame available through Soiltest, Inc., but with dimensions increased to permit testing of large triaxial specimens. Loads were applied as dead weight increments to the hangers at the underneath side of the frame. The rear hanger provided a 10:1 load ratio to the piston and was used exclusively for providing weight to balance the cell pressure exerted on the piston. The forward hanger permitted a 40:1 load ratio to the piston. This hanger was used for all load increments. Counterbalances within the lever system insured that no loads from the frame itself were applied to the specimen during the test operation. A mechanical jacking device was utilized to maintain the loading arm horizontal during testing regardless of the deformation of the specimen. Load increments of a nominal 50, 20, and 10 psi to a four inch diameter specimen were fabricated from plate steel for use in load

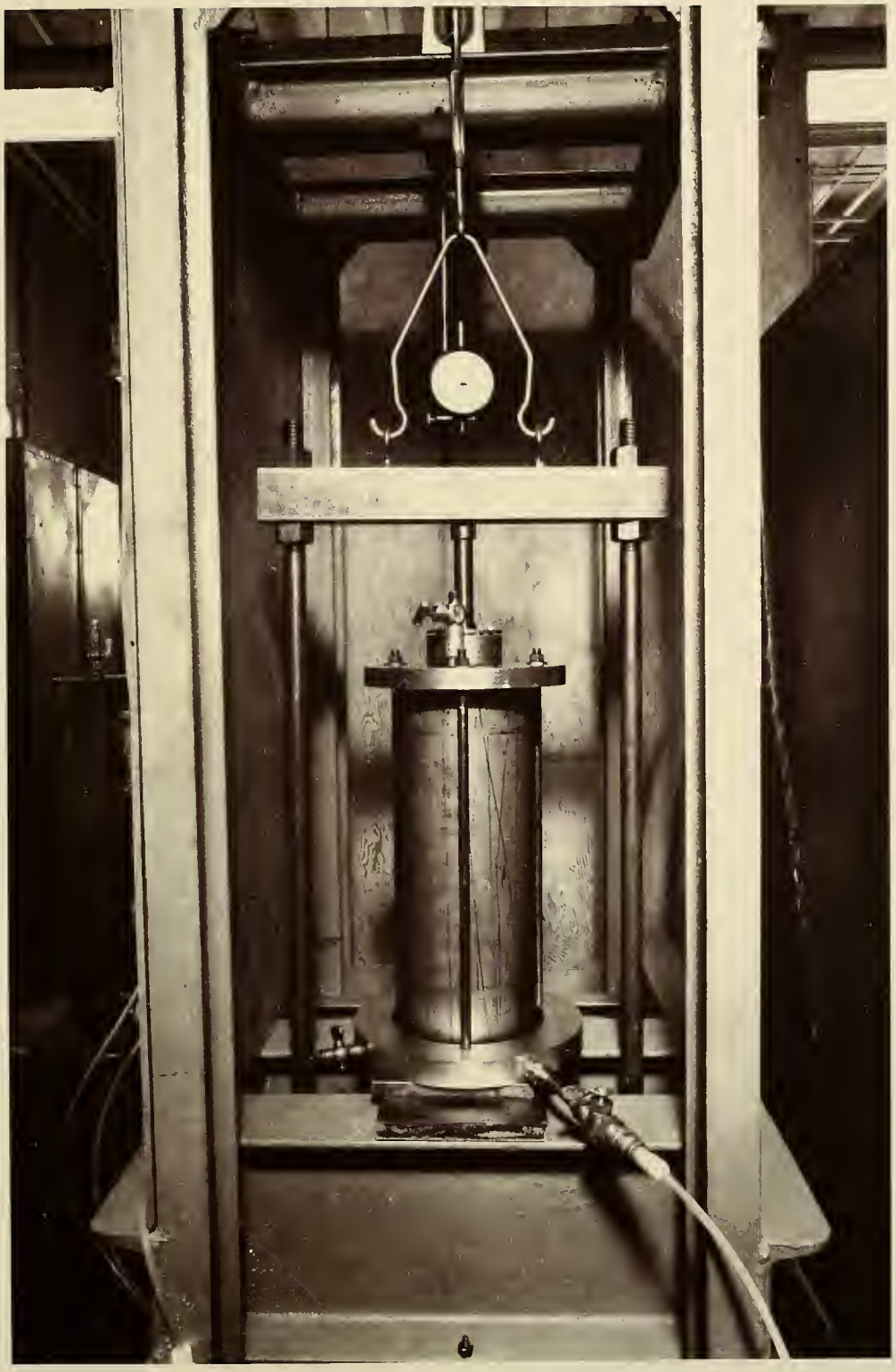


FIGURE 9. TRIAXIAL CELL IN LOADING FRAME

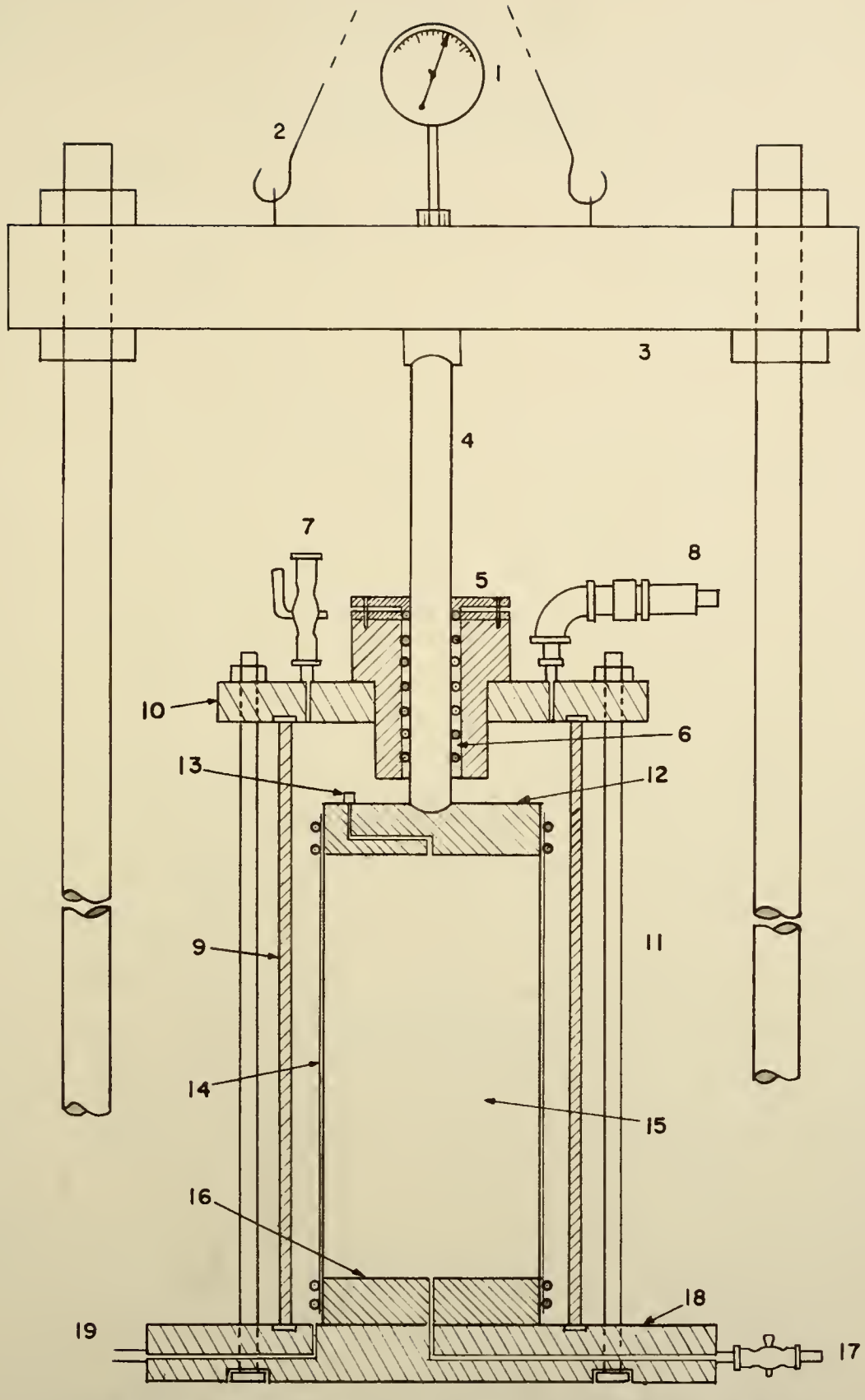


FIGURE 10. DETAILS OF TRIAXIAL CELL

TABLE 6

Key to Details of Figure 10

1. Dial gage, 0.01 mm.
2. Loading head support
3. Loading head
4. Piston
5. O-Ring seal assembly
6. Ball-bushing seal assembly
7. Air escape valve
8. Pressure relief valve
9. Cylindrical wall of triaxial cell
10. Upper head assembly
11. Tie rods
12. Upper loading cap
13. Upper drainage connection (sealed)
14. Rubber membrane with O-Ring seals
15. Specimen
16. Bottom loading cap
17. Bottom drainage connection with valve
18. Bottom head assembly
19. Water inlet connection

application. Axial deformation of the specimen was measured with a 0.01 millimeter dial gage suspended from a top crossbar of the frame and with the gage pointer in contact with the loading crossbar and positioned over the center of the piston.

The triaxial cell was modified from one available in the Bituminous Mixtures Laboratory from previous work done by Oppenlander (4). The top and base of the cell were of aluminium and the cylinder was brass. Compression loads were applied to a loading cap resting in position on the specimen by means of a hardened steel piston. The loading cap was recessed to receive the spherical end of the piston. The piston was centered through a vertical ball bushing and grease seal in the center of the cell. Preliminary tests showed the grease seal unable to hold the higher lateral pressures without appreciable leaking. Consequently, an O-Ring seal was installed above the grease seal. Adjustment of thumb screws permitted a cover plate to be forced against the O-Ring which expanded against the piston to provide a water-tight seal. The remainder of the cell details are apparent upon reference to Figure 10 and to the identification key shown in Table 6.

The use of the O-Ring seal was successful in eliminating leakage but it was apparent that this procedure materially increased frictional resistance to the piston. In order to evaluate piston friction, a proving ring was set up inside the cell which was assembled using a clear lucite cylinder. The use of known loads on the hanger permitted a computation of the load applied to the piston. Evaluation of the load reaching the proving ring was possible through the calibration charts of the proving ring. The difference between these loads was

assumed to be friction. A plot of load applied versus friction was developed and used to correct all applied loads in the computation of test results. This plot is presented in Appendix D.

Triaxial Test Procedure

All testing was performed using the 'open' or constant-lateral-pressure triaxial test procedure. The following sub-sections describe the details of the preparation of specimens for testing, the loading procedures used, the technique utilized in disassembling the apparatus upon completion of a test, and an evaluation of the test procedure.

Preparation of Specimens for Testing

The preparation of a specimen for testing was a time-consuming operation requiring careful attention to details. The specimen to be tested was removed from storage in the controlled-temperature room and carefully centered on the bottom loading cap of the triaxial cell. The top loading cap was placed on the sample. The circumferential surfaces of both caps were covered with a thin film of Dow-Corning Hi-Vacuum Silicone Grease. A thin rubber membrane, available from Soiltest, Inc., in a size adequate for four inch diameter by eight inch high specimens, was placed over the specimen utilizing a membrane stretcher for the operation. The membrane was smoothed carefully to the surface of the specimen and both loading caps. The membrane was sealed to the loading caps by the use of two O-Rings (2-7/8 inch diameter) at each end. A vacuum was applied to the specimen through the lower drainage connections. If the membrane held the vacuum it was assumed to be a leak-free set-up and the assembly of the cell continued. A photograph of a

partially completed test assembly is shown in Figure 11.

This procedure was used for all samples except those to be tested with a 60 psi confining pressure. Experience indicated that one membrane was not capable, in general, of withstanding this level of confining pressure without rupturing or leaking. As a result, double membranes were used for all 60 psi confining pressure tests. The second membrane was placed over the first one and secured by an additional two O-Rings at each end. The double membrane procedure performed well for the high-confining-pressure tests.

The cell assembly was completed by putting the cell cylinder, the connecting rods, and the top including the piston into place. The connecting rods were fastened by tightening the rod nuts uniformly and the piston was pressed firmly into place in the spherical seat of the upper loading cap. The assembled cell and specimen were placed in position within the triaxial frame.

Once the cell and specimen were placed in the loading frame the test progressed in a routine manner. The loading bar was brought into contact with the piston and adjusted to a balance in this position. The hanger loading arm was leveled and the deflection dial adjusted in place on the loading bar. The volume measuring device was attached to the water inlet and the cell was filled with water. During the filling operation, the valves to the measuring tube and mercury reservoir tube were closed. Gravity flow was not sufficient to fill the cell; therefore, a slight (5 to 8 psi) pressure was applied to the top of the reservoir tank. When water first flowed from the air escape valve, the water inlet valve and the water reservoir outlet valves were closed and

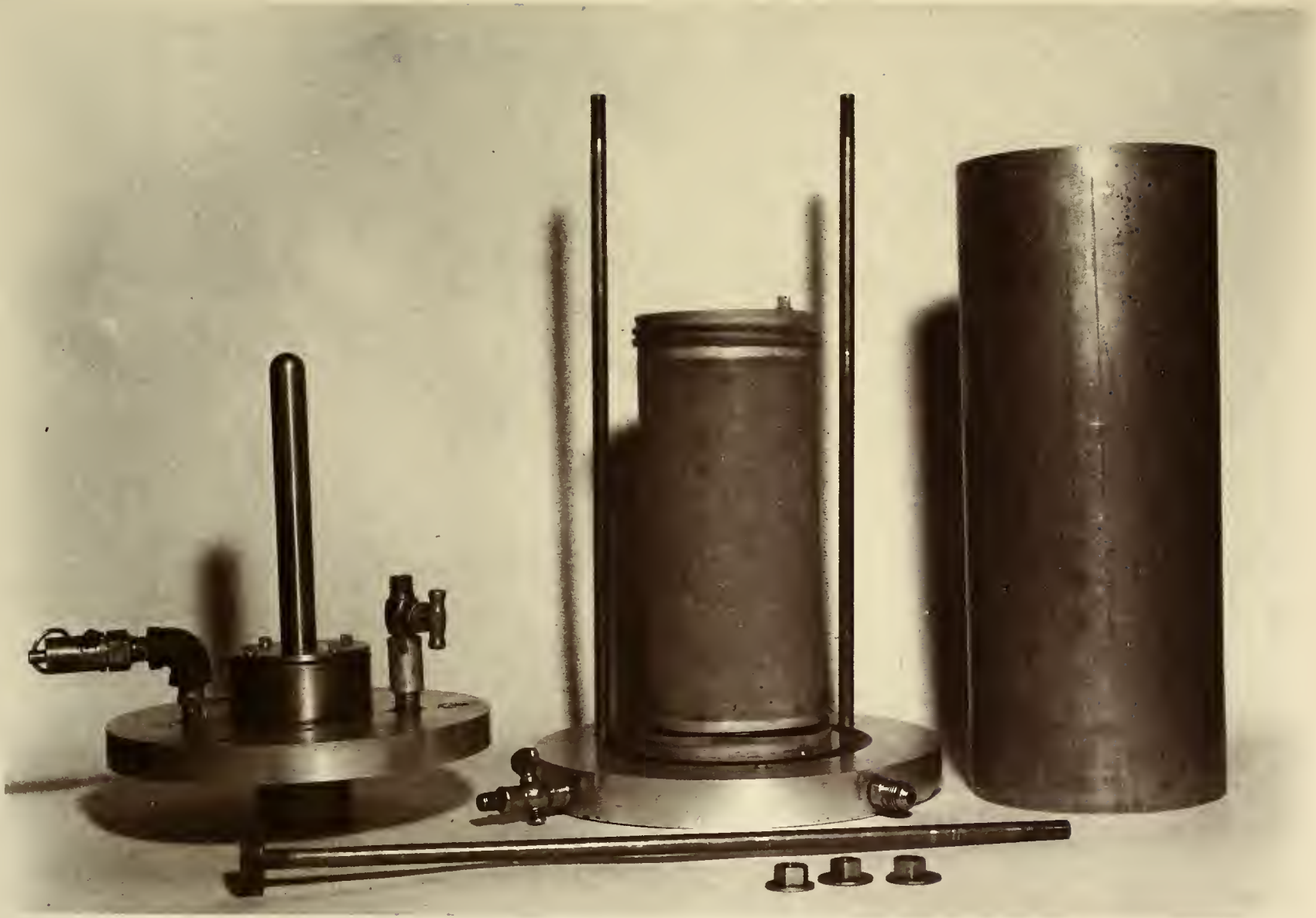


FIGURE 11. ASSEMBLY OF TRIAXIAL CELL

the pressure connection to the water reservoir was disconnected. To check on the removal of all air from the cell, the water reservoir tank was manually elevated, the outlet valve opened, and the cell water inlet valve opened until air-free water flowed from the air escape valve. The two water valves were then closed and the reservoir tank returned to its support. With the cell full of water, the air escape valve and the bypass valve were closed. The cell water inlet valve and the valves to the mercury reservoir and the measuring tube were opened.

Loading Procedure for Routine Tests

With the completion of the above steps, the specimen was ready to test. An initial reading of the level of the mercury in the measuring tube and a deformation dial reading were obtained. The specimen drainage valve was adjusted in accordance to the type of test, closed for the "Quick" test, and open for all other types of tests. The desired confining pressure was applied to the water reservoir with adjustment of the pressure accomplished through the control valve on the nitrogen tank. To start the test, the water reservoir outlet valve was opened permitting the test pressure to be applied to the system while simultaneously a weight was placed on the 10:1 load hanger to balance the cell pressure acting on the loading piston.

Regardless of the type of test being performed, readings of the volume measuring tube and deformation dial were obtained at 15 seconds after the application of the confining pressure. The 15-second reading was utilized during the computation stage as will be discussed under that heading.

In the performance of a "Quick" or "Q" test, a reading of both the volume measuring device and deformation dial was obtained at 5 minutes after the application of the confining pressure. At this time the first axial load was applied. Readings of the volume-measuring device and deformation dial were taken at 5-minute intervals just prior to the application of the next axial load increment. In the interval between applications of axial loads the leveling device was manipulated to maintain the hanger loading bar as close to horizontal as possible. Axial loads were applied in this fashion until failure of the sample occurred. Failure was considered to have taken place when (a) axial deformation exceeded one centimeter, (b) the leveling adjustment could not maintain pace with deformation and/or (c) the amount of volume change was so great as to present a danger of forcing the mercury level beyond the limits of the volume measuring device.

If "Consolidated Quick" or "Consolidated-Partially Drained" tests were being performed, readings were made of the volume measuring device and the deformation dial at times of 0.25, 0.5, 1, 2, 4, 8, 15, 30, and 60 minutes after the application of the confining pressure. A plot of the volume measuring device readings versus time (to a logarithmic scale) was prepared as the readings were observed. Full compaction of the specimen under the applied confining pressure was assumed to have occurred if the 60 minute reading plotted as a straight line with the preceding points at a slope appreciably less than that defined by the plotted points at earlier times. This process is similar to that used for "time curves" for various load increments in consolidation tests of soils.

For "Consolidated Quick" or "Q_c" tests, the specimen drainage valve was closed at the 60-minute reading after the application of the lateral pressure. Axial load increments were applied and readings observed in the same way and at the same rate as for the previously described "Quick" tests. For "Consolidated-Partially Drained" or "CP" tests, the tests were completed in exactly the same fashion as for the "Q_c" tests except that the specimen drainage valve remained open throughout the test.

Tests at the three drainage conditions described were performed at confining pressures of 15, 30, and 60 psi on specimens made at 4 and 5 percent asphalt contents by weight of mixture and 150 and 400 psi compaction pressures. "Quick" tests at the same three confining pressures were performed on specimens prepared at 3, 4, and 5 percent asphalt contents by weight of mixture with compaction pressures of 250 and 500 psi. Additional "Quick" tests were performed on specimens fabricated at 150 and 400 psi compaction pressures at an asphalt content of 3 percent by weight of mixture and using the specified confining pressures. Typical data and computation sheets are presented in Appendix B.

Slow Rate of Loading Tests

Three tests were performed at a confining pressure of 30 psi and using an interval of one hour between the application of axial load increments rather than the 5-minute interval used for the routine tests.

The specimens were prepared in the usual fashion at a single asphalt content and compaction foot pressure. One specimen was tested at each of the drainage conditions represented by the "Q", "Q_c" and "CP" types of test. With the exception of the rate of axial load increment

application, the tests were performed in exactly the same manner as the routine tests.

Water Saturated Tests

Two tests were performed after water saturation of specimens which had been compacted to a given set of initial conditions of asphalt content, compaction foot pressure, and air voids.

The water saturation was accomplished by placing the specimen in an air-tight chamber and subjecting it to a vacuum for a period of 30 minutes. Without releasing the vacuum, water was permitted to enter the chamber and cover the specimen. After an elapsed time of 30 minutes, the vacuum was applied again to the chamber and maintained for an additional half-hour. The application of the vacuum was discontinued and the specimen permitted to remain in the water within the evacuated atmosphere overnight or for a period of approximately 16 hours.

Upon removing the specimen from the chamber, it was measured for volume computations and weighed in a surface-dry condition to establish the amount of water that had been absorbed.

Testing was accomplished using a "Q" test at confining pressures of 15 and 60 psi. The cell was modified for these tests to insure no drainage by introducing solid aluminium loading caps at the top and bottom of the specimen. The solid loading caps replaced ones which contained grooves to facilitate drainage and were provided with drainage outlets. The solid loading caps were used only for the water saturated tests. In all other respects the testing procedure was similar to that previously described for the "Q" test.

Disassembly of Test Apparatus

At the completion of the test, the mercury reservoir and volume measuring tube valves were closed and the bypass valve opened. Pressure was released from the water reservoir tank, the cell air-escape valve was opened, and the cell liquid drained into the water reservoir tank. The deformation dial was moved clear of the loading crossbar and all load increments removed from the hangers. The cell water-inlet valve was closed before the cell was completely drained to prevent air from entering the plastic tubing system. The volume-measuring assembly was disconnected from the triaxial cell and the water remaining in the cell recovered in a glass cylinder for return to the water reservoir tank. The cell was removed from the controlled-temperature room for disassembly.

Upon removal of the brass cylinder from around the specimen, the specimen was inspected carefully for membrane leaks that may have occurred and would invalidate results. The membrane was stripped from the specimen and the cell cleaned thoroughly in preparation for the next test.

Evaluation of Test Procedure

Three problems were paramount in the triaxial testing procedure outlined above. The first was associated with the membranes. Occasional failures occurred and were apparent by leakage of liquid from the specimen drainage valve for tests where this valve was open, by continued unusual movements upward of the mercury level of the volume measuring tube, or by inspection of the membrane and specimen at the end of the test. In cases where such failures occurred the test

results were disregarded and a new test performed. A second problem was leakage of the equipment during the test. Close observation of all seals, joints, etc. was maintained during all tests. If care was exercised in the set-up, no leaks occurred. The last problem involved the change in volume of the cell and the volume measuring equipment under the application of the confining pressures. This problem was solved by periodic calibration tests made under all test confining pressures and by using a steel specimen. The volume changes observed during calibration trials took place almost instantaneously and were recorded for a correction of the 15-second readings taken during tests. As all other test data involved the use of differences in successive readings any errors in equipment that may have been introduced in a particular test were contained within the 15-second reading. Calibration values for the confining pressures used in the reported tests are presented in Appendix D.

All equipment performed creditably. The procedures outlined above proved to be both reasonable and adequate though they were time-consuming and occasional membrane failures, leaks, etc. were discouraging.

RESULTS AND COMPUTATIONS

This section serves to summarize the observed test data, to present the computations used, and to provide a statement of the results of the tests and computations. The section is divided into sub-sections based on the major areas of the study.

Observed Data

The compacted specimens were weighed to nearest one-half gram and the diameter and height determined as an average of at least three measurements. All linear measurements were made to the nearest one-hundredth of a centimeter. These measurements provided the initial height and volume values of the specimen upon which all computations were based.

The weights of aggregate and asphalt recorded for each mixing operation permitted the computation of the average asphalt content by weight of mixture for each specimen. The measured weight, volume and average asphalt content of a specimen, together with the average bulk specific gravity of the aggregate blend, the specific gravity of the asphalt, and the percent absorption of asphalt by the aggregate made it possible to compute the volumes of aggregate, asphalt, and air in the specimen.

During the process of performing each triaxial test, readings were taken of the extensometer used to measure the change in height of the specimen and of the level of the mercury in the volume measuring tube

at predetermined intervals. These readings permitted the axial strain to be computed for each load increment and provided a means of establishing the change in volume of the specimen under the various loads.

A typical mix record, initial characteristics computation, and test data sheets and computations are shown in Appendix B.

Stress-Strain Relationships

Loads were applied to the test specimen in multiples of a nominal 10 psi to a four inch diameter specimen. A correction to the applied load was made for the friction developed between the piston and the cell. Thus, the total load applied to the specimen was known for each increment of load.

Preliminary tests indicated that volume changes occurred under load to such a magnitude that adjustment of the specimen area was required for the computation of applied stress. This adjustment was made on the assumption that even though the specimen changed volume it would retain a uniform cylindrical shape. This is not strictly the case; however, it is felt that this assumption provided the most reasonable method of computing the average stress acting on the specimen under any given load increment. The average areas were computed in the following fashion:

(1) The initial volume of the specimen was adjusted in accordance to the volume change that occurred under any load as indicated by the volume measuring device.

(2) The height of the specimen under the same load was obtained by subtracting the change in height from the initial height of the specimen.

(3) The average cross-sectional area of the specimen was computed as the quotient of the specimen's volume divided by its height.

Axial strains were determined by dividing the change in height of the specimen by its initial height. In some cases there were slight upward movements of the extensometer during the period of application of the confining pressure only. Strain computations ignored these movements and were based on a zero dial reading obtained at the start of the test.

Typical stress-strain curves are illustrated in Figure 12 for one of the routine test series. Figure 13 shows similar data for one of the three slow rate of loading tests. All tests resulted in curves of a similar shape. In particular, the curves followed a straight line from zero deviator stress to a point designated as the Proportional Limit (P.L.). Beyond the P.L. the stress-strain curve deviated from the straight line at a rate that increased with axial strain. In approximately one-half of the tests, a peak stress value was obtained which was followed by a decrease in stress with further strain. In tests having this characteristic, pronounced shear planes were observed in the specimen. In the remainder of the tests a bulging of the specimen was noted and a typical plastic failure stress-strain curve resulted. Figure 14 shows a typical specimen before testing and representative failures. The failure planes shown in this figure have been emphasized in order to be visible in the photograph.

All tests were stopped at approximately five percent strain. At this point, the specimens had either developed a shear failure or the

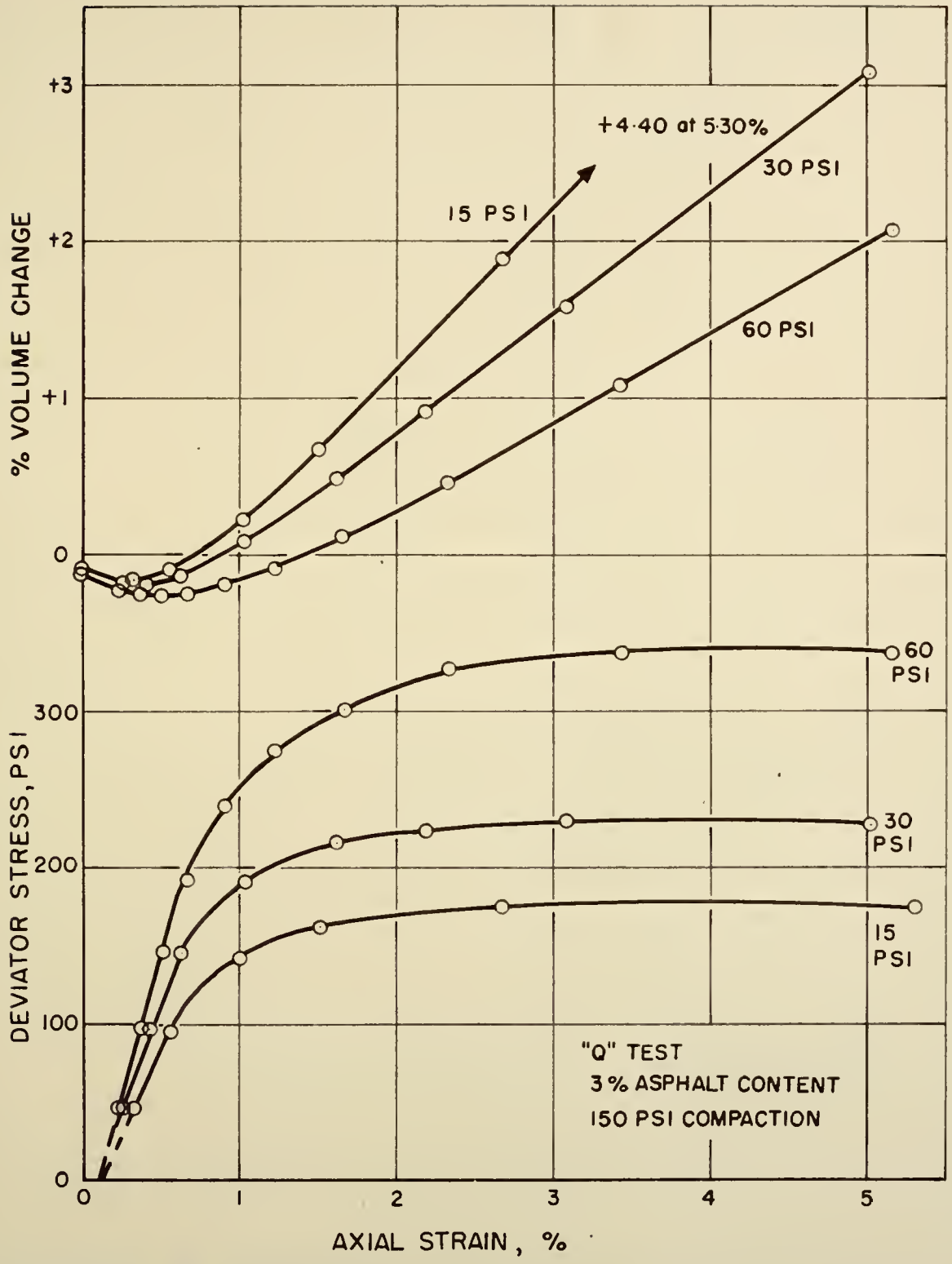


FIGURE 12. STRESS AND VOLUME CHANGE VS STRAIN

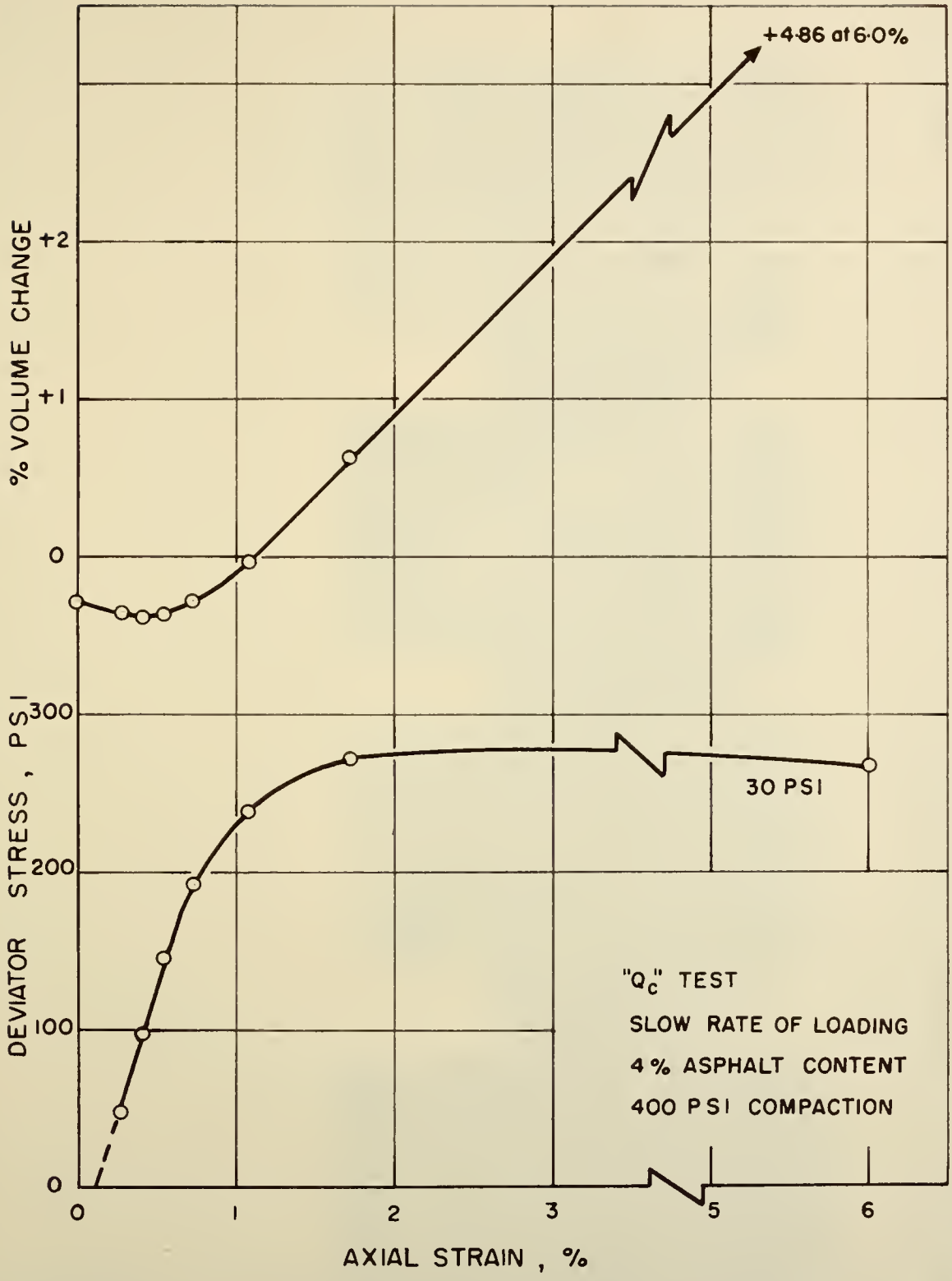


FIGURE 13. STRESS AND VOLUME CHANGE VS STRAIN – SLOW RATE OF LOADING

FIGURE 14 TYPICAL SPECIMENS BEFORE AND AFTER TESTING

stress-strain curve slope for bulging failures was so flat as to constitute failure for practical purposes.

Table 7 (Appendix A) presents a summary of the data obtained from all tests. It includes sufficient stress and strain values that a reasonable facsimile of the stress-strain curve for any test may be sketched.

Modulus of Deformation

The Modulus of Deformation (C_d) is defined for the purposes of this study as the deviator stress at the Proportional Limit divided by the axial strain at that point. The strain used for this computation was corrected for the intercept of the straight line portion of the stress-strain curve with the strain axis. The term, Modulus of Deformation, is utilized in accordance with common practice in the design of flexible pavements (68).

Figure 15 presents a plot of the computed C_d values for the various confining pressures for combinations of 4 and 5 percent asphalt content and 150 and 400 psi compaction foot pressures. Table 8 (Appendix A) presents the data from which Figure 15 was developed and Table 9 (Appendix A) contains a summary of the pertinent information for tests at other combinations of asphalt content and compaction pressure and for the special tests that were performed.

Strength and Stability Determinations

As stated in the Review of Literature, the question of how to establish strength and/or stability constitutes a major problem in the testing of asphaltic mixtures. For purposes of this study, deviator stresses at four points are reported: (a) at the Proportional Limit,

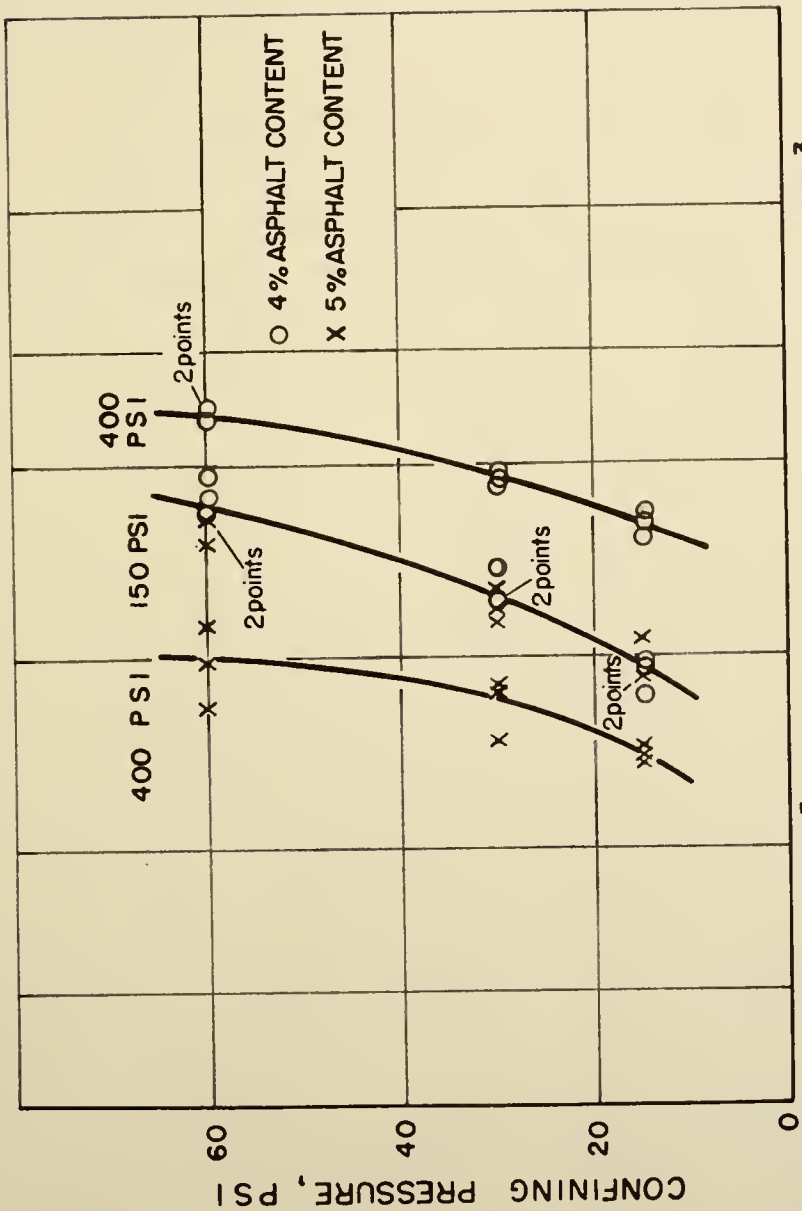


FIGURE 15. MODULUS OF DEFORMATION VS CONFINING PRESSURE

(b) at an axial strain of two percent, (c) at failure or the best estimate of the failure stress where this condition is not clearly established by the stress-strain curve, and (d) at the point where the volume change of the sample under load was zero.

The results of the strength and stability determinations are presented in Figures 16, 17, 18, 19, 20, and 21 as plots of vertical stress against lateral stress for conditions of failure, two percent strain, and at the point of zero volume change for samples prepared at combinations of 4 and 5 percent asphalt content and 150 and 400 psi compaction pressure. Deviator stress data for the same conditions for all other tests are tabulated in Table 7 (Appendix A). A typical Mohr's circle plot for one test series is shown in Figure 22 for the failure condition and in Figure 23 for stresses at two percent axial strain.

Figures 24 and 25 indicate the variation in deviator stress at failure with asphalt content for the various compaction pressures. Also shown in these figures are plots of the variation in bulk density and aggregate density with asphalt content for the different compaction pressures. Data from all routine tests are shown in these figures. For combinations of compaction pressure and asphalt content at which more than one test was performed, the average values of compressive strength, bulk density, and aggregate density are shown in Figures 24 and 25.

Figure 26 presents a graphical summary of the values of the observed shear strength parameters, "c" and "Ø", at failure for the various asphalt contents and compaction pressures. Average values of the parameters are plotted for conditions at which more than one test series

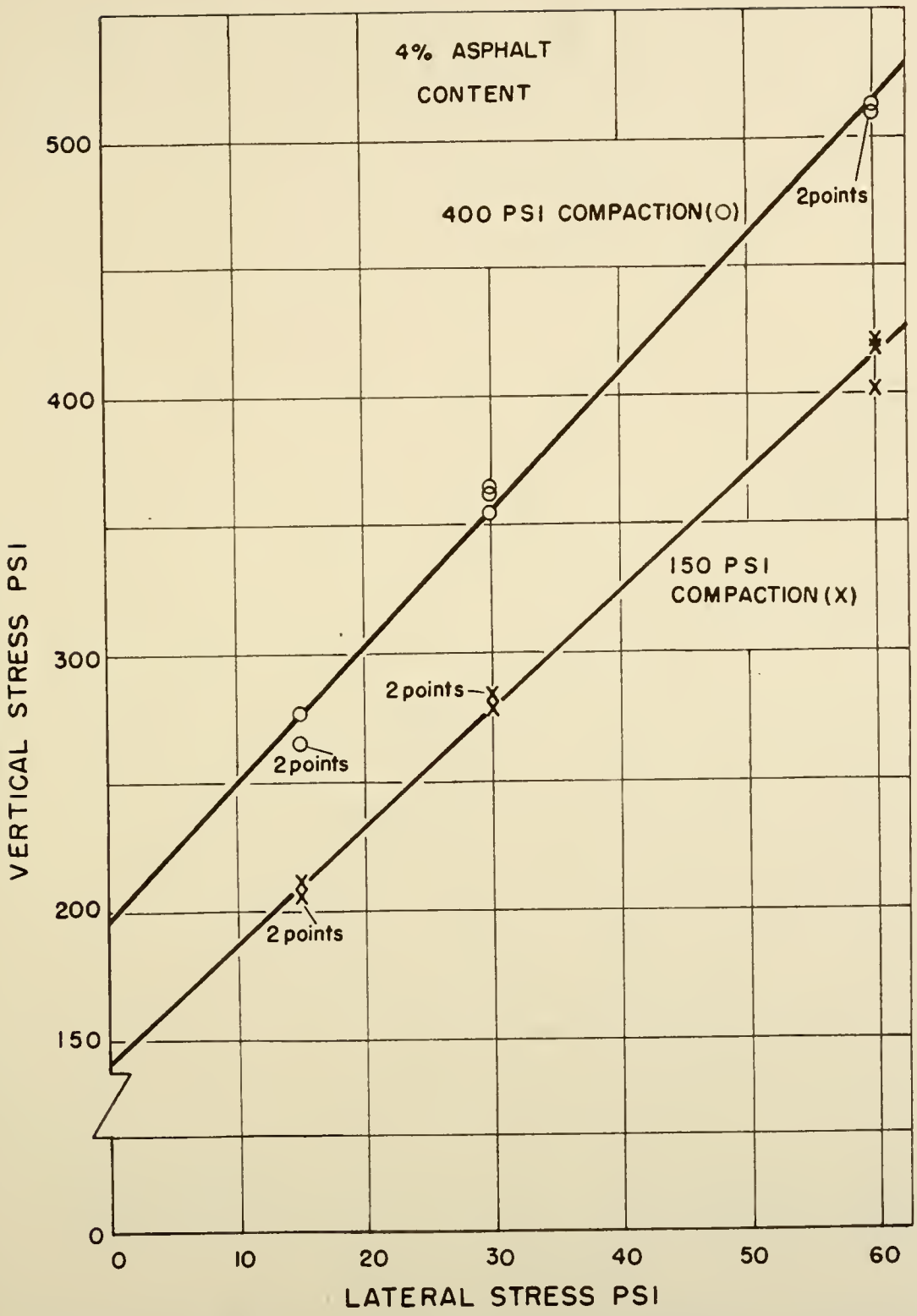


FIGURE 16. VERTICAL STRESS VS LATERAL STRESS AT FAILURE – 4% ASPHALT CONTENT

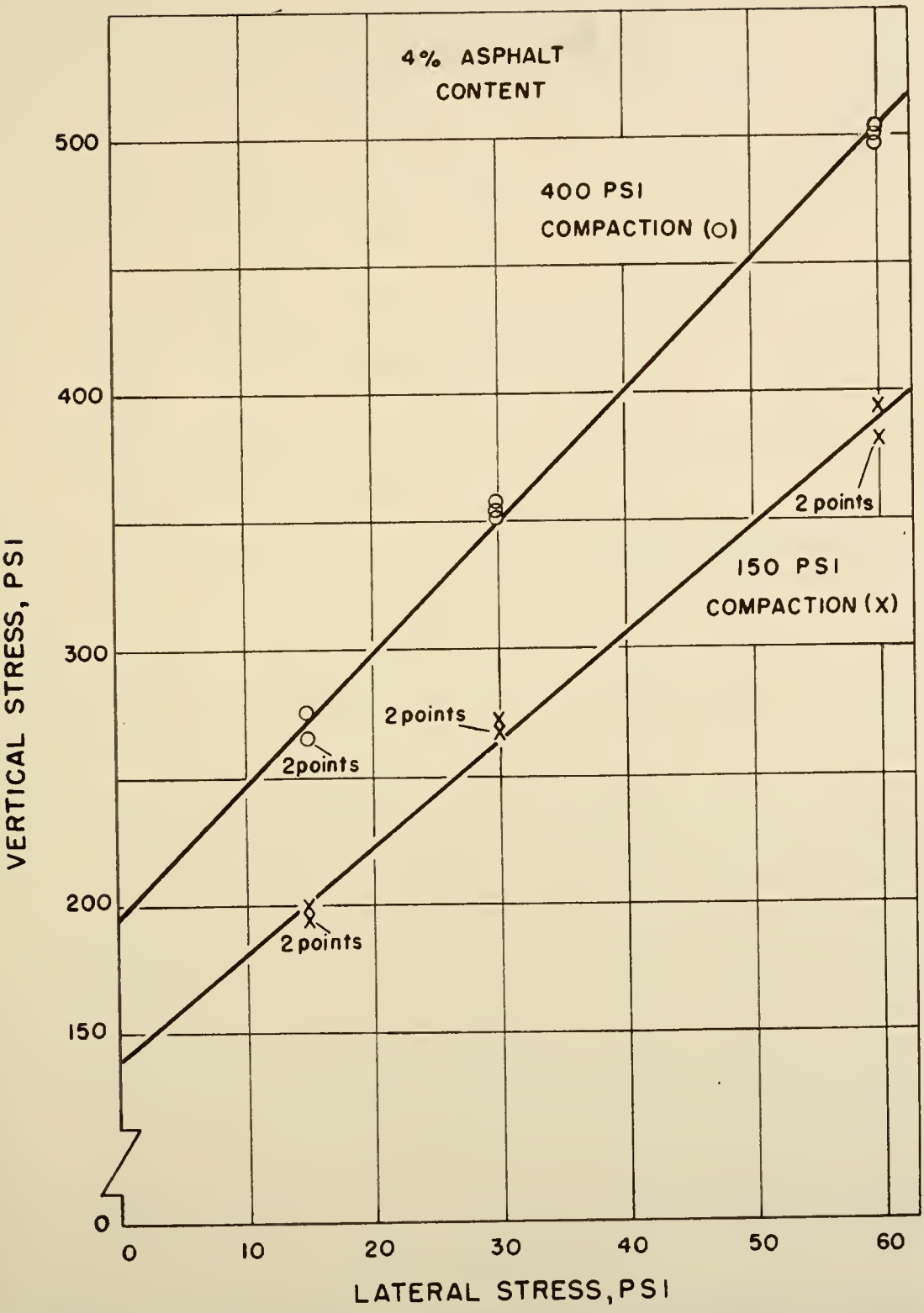


FIGURE 17. VERTICAL STRESS VS LATERAL STRESS AT 2% STRAIN – 4% ASPHALT CONTENT

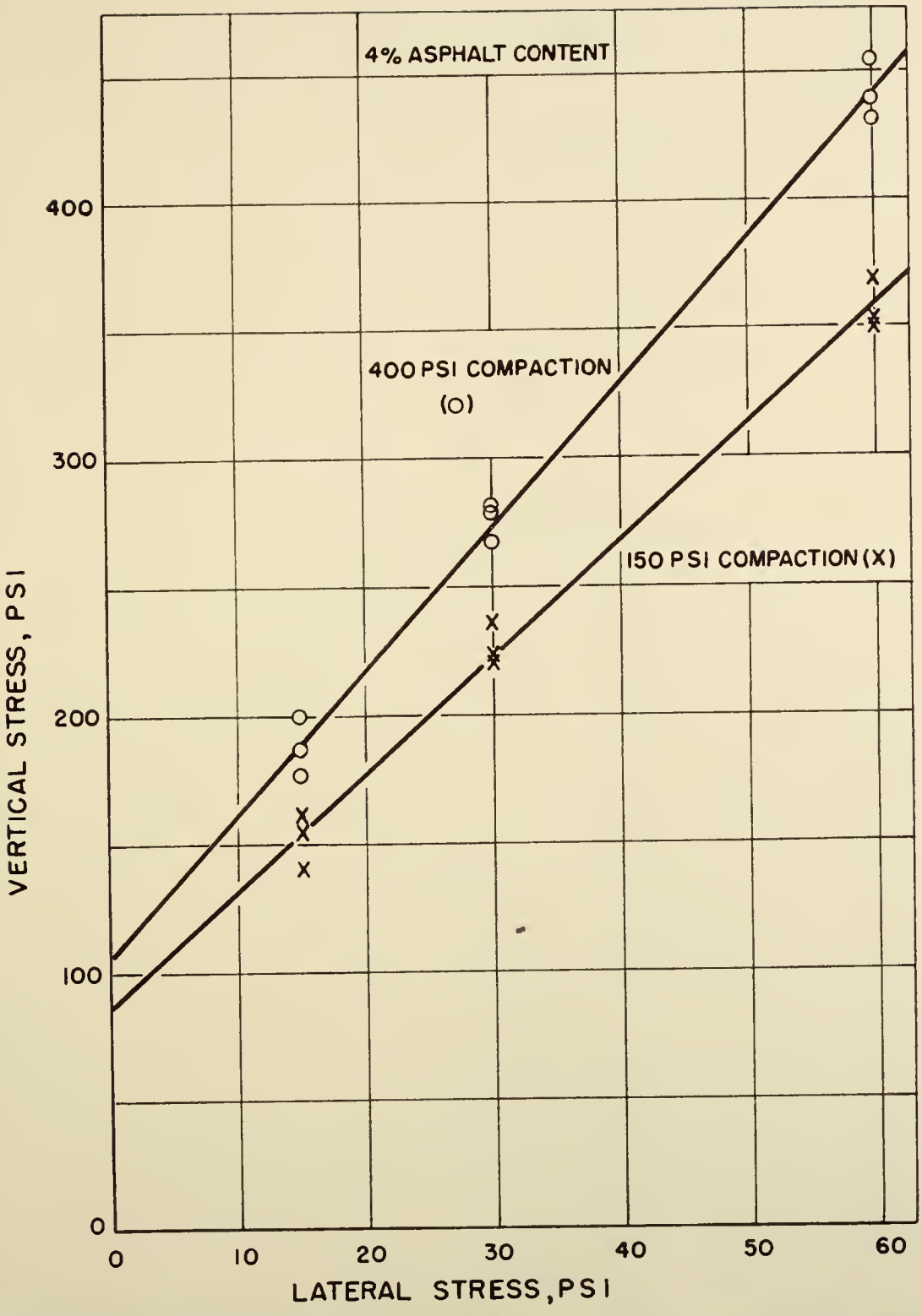


FIGURE 18. VERTICAL STRESS VS LATERAL STRESS AT 0% VOLUME CHANGE – 4% ASPHALT CONTENT

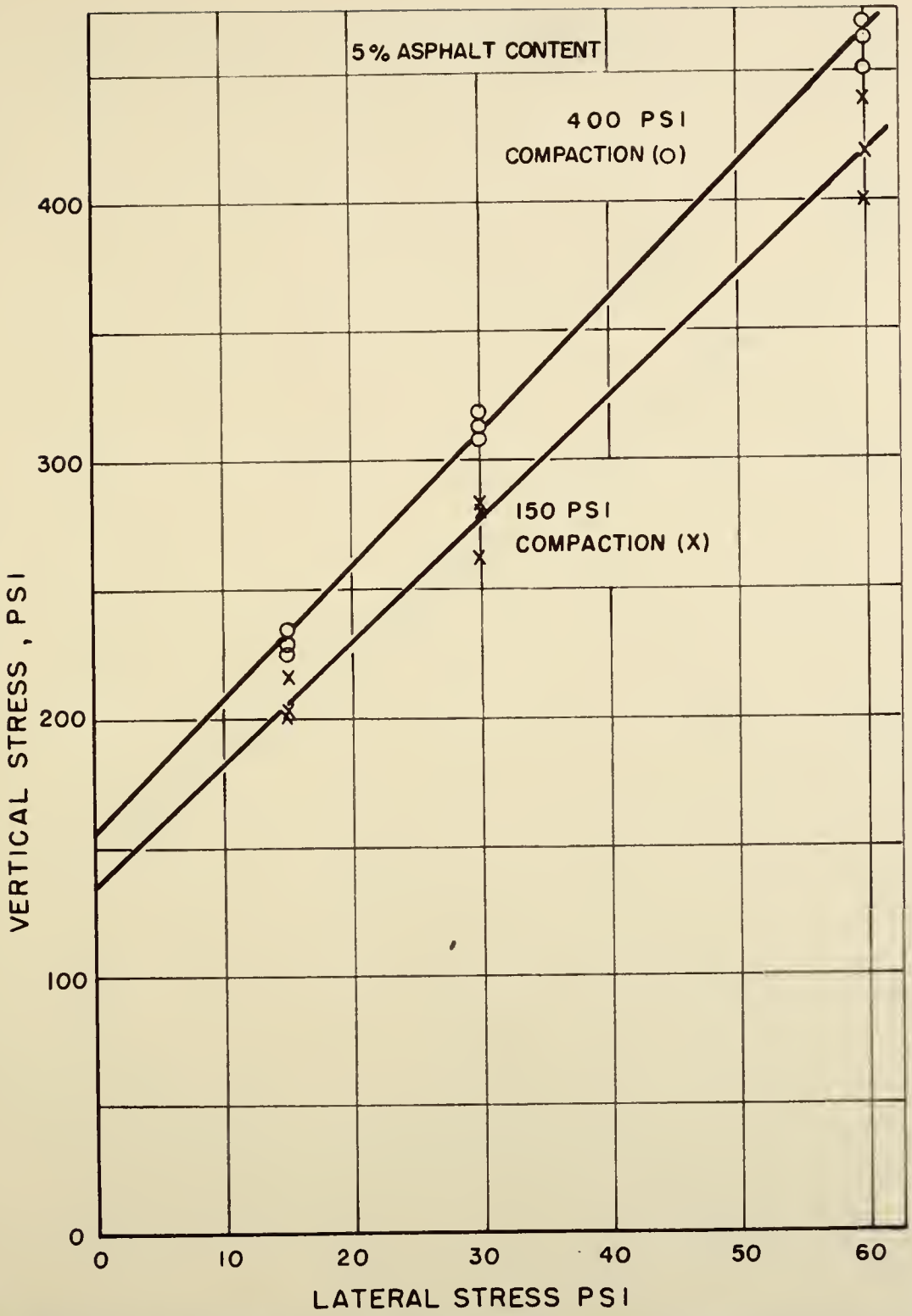


FIGURE 19. VERTICAL STRESS VS LATERAL STRESS AT FAILURE – 5% ASPHALT CONTENT

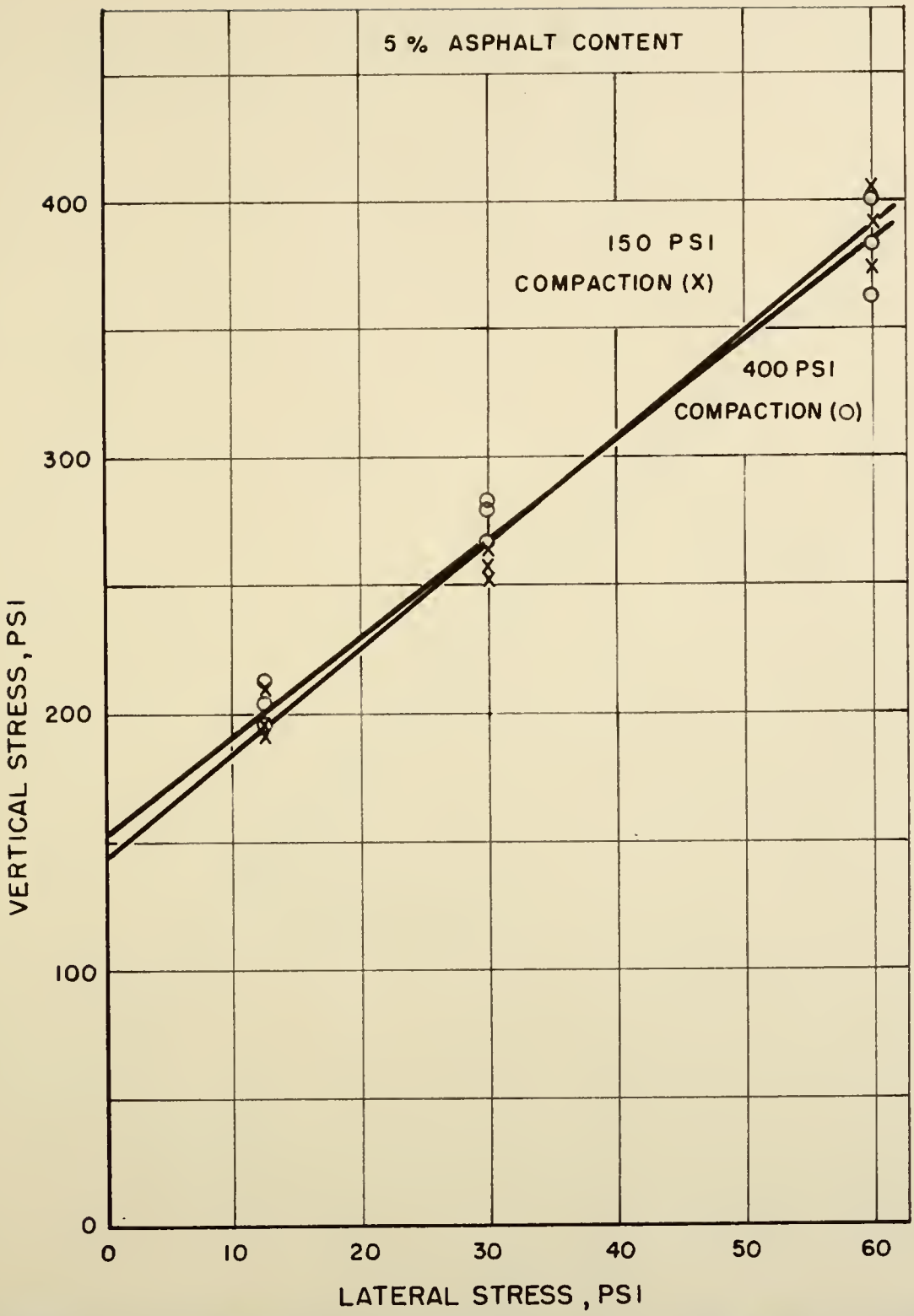


FIGURE 20. VERTICAL STRESS VS LATERAL STRESS AT 2% STRAIN – 5% ASPHALT CONTENT

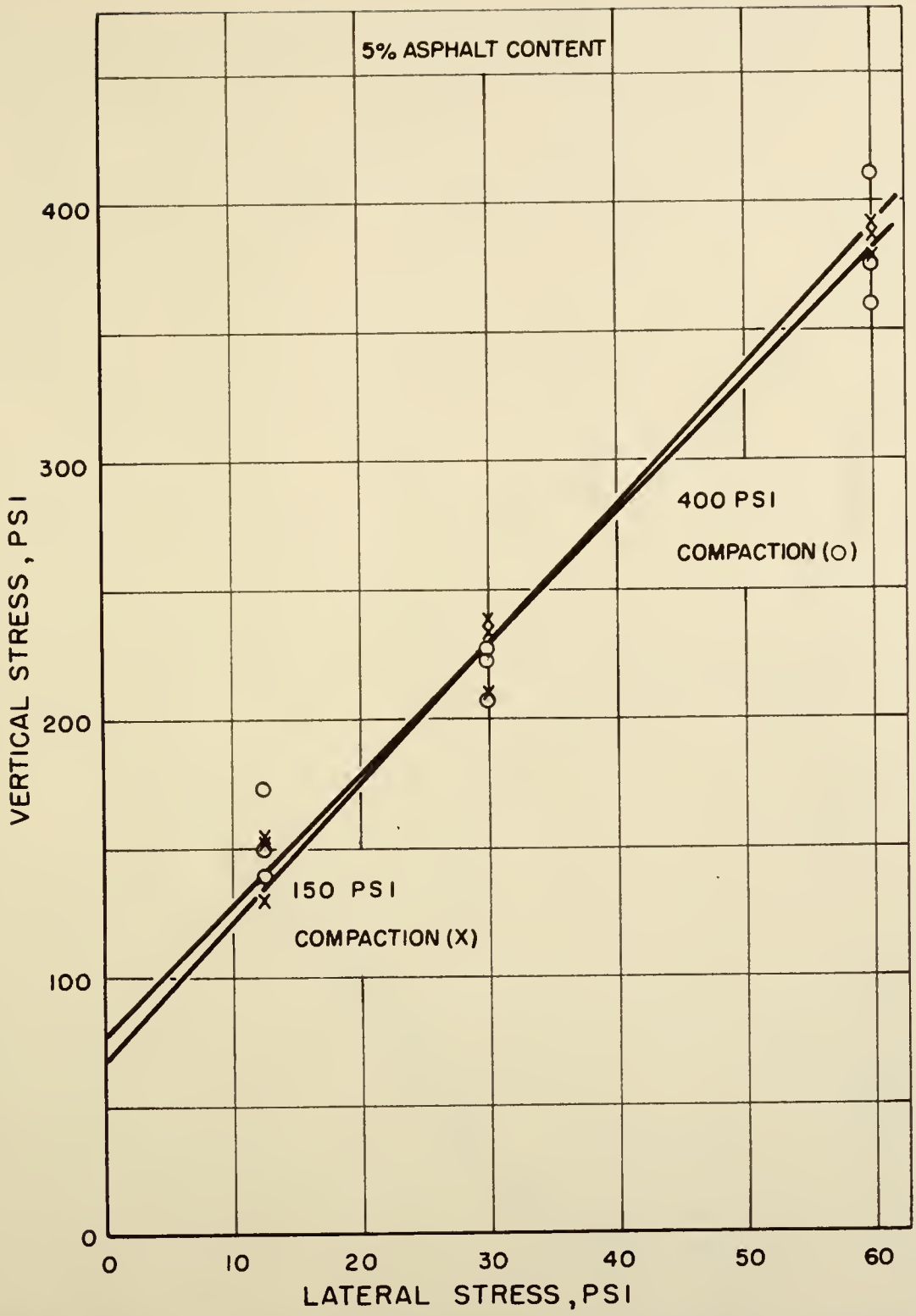


FIGURE 21. VERTICAL STRESS VS LATERAL STRESS AT 0% VOLUME CHANGE – 5% ASPHALT CONTENT

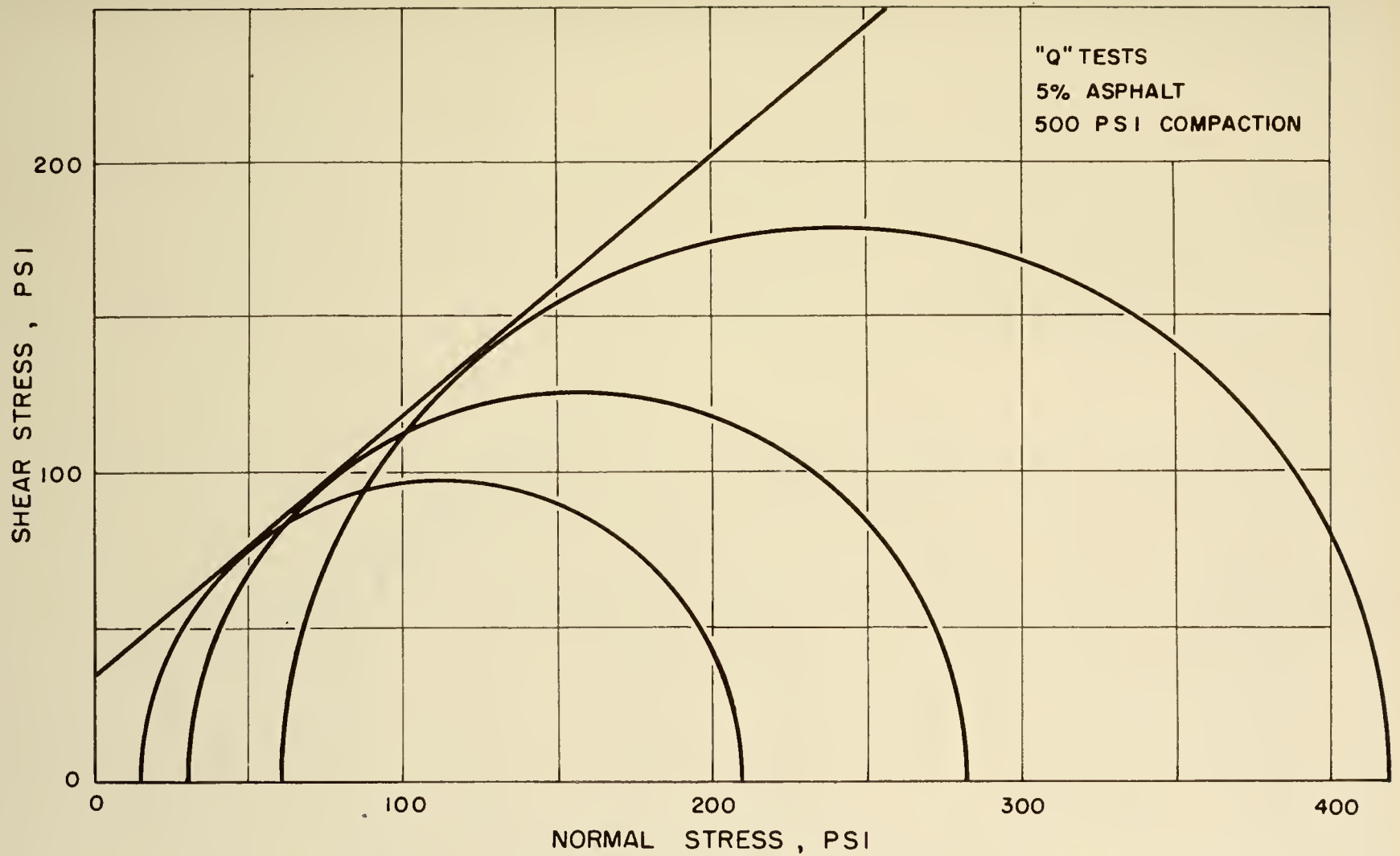


FIGURE 22. CIRCLES OF STRESS AT FAILURE

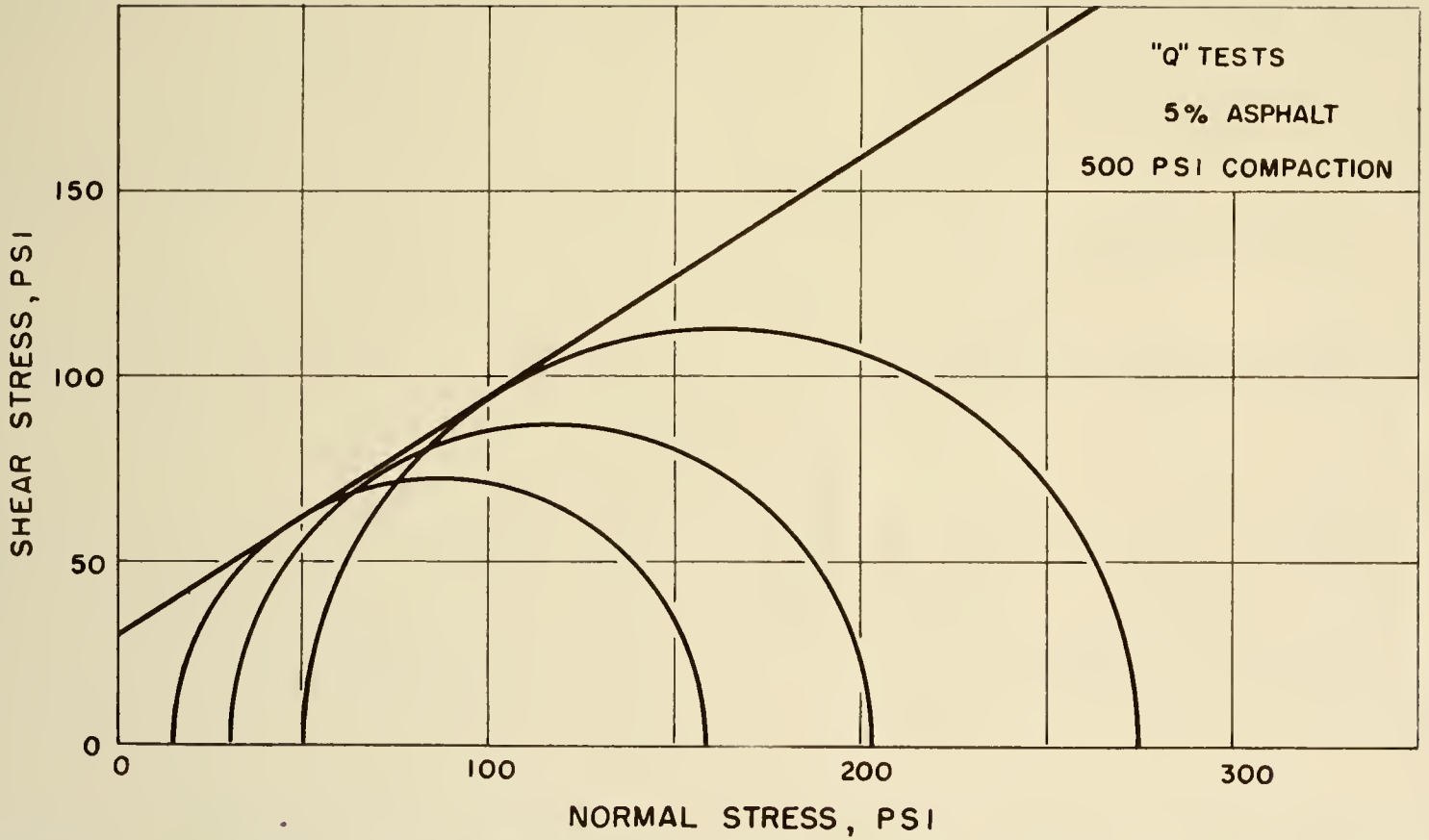


FIGURE 23. CIRCLES OF STRESS AT 2% STRAIN

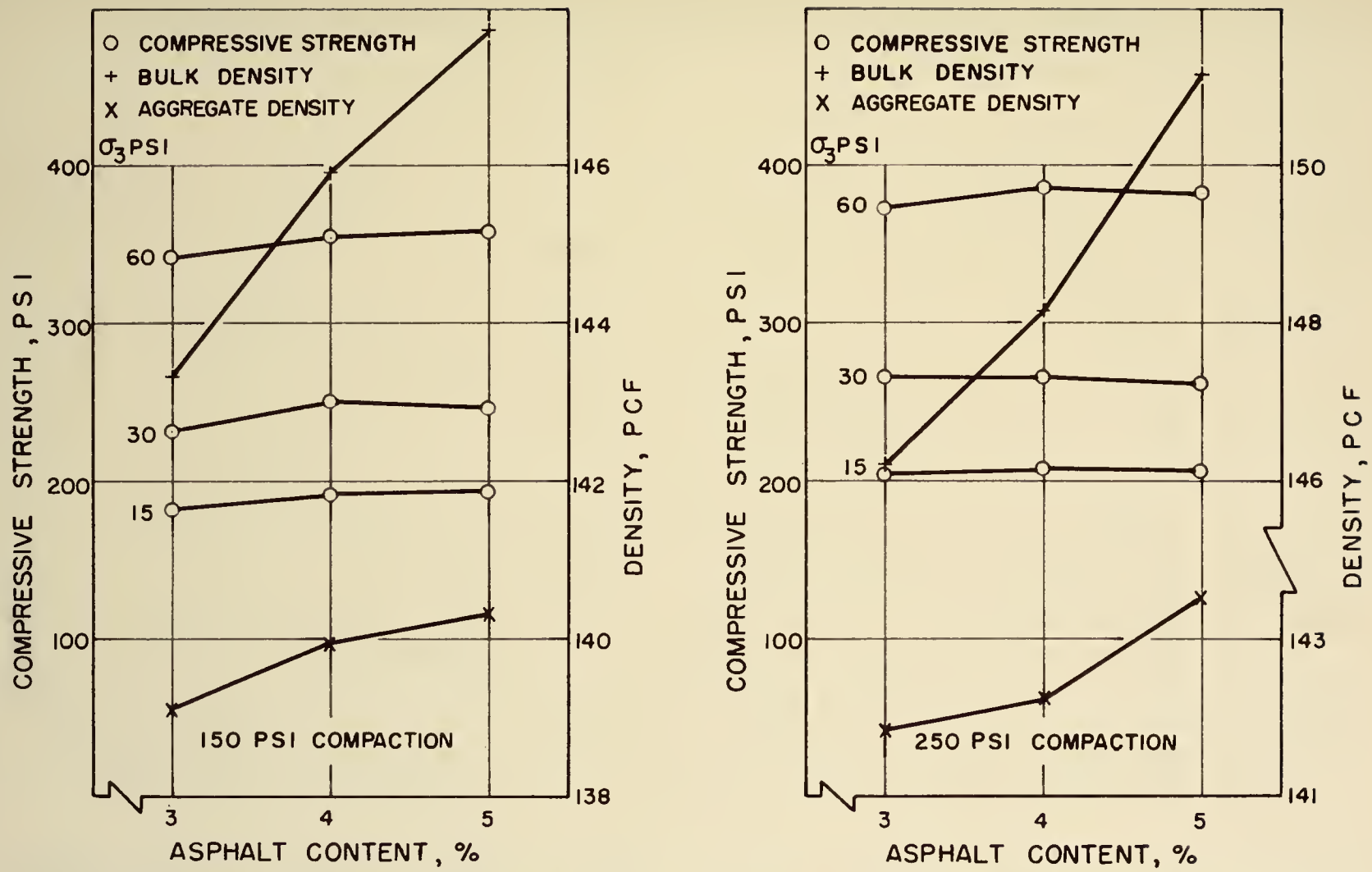
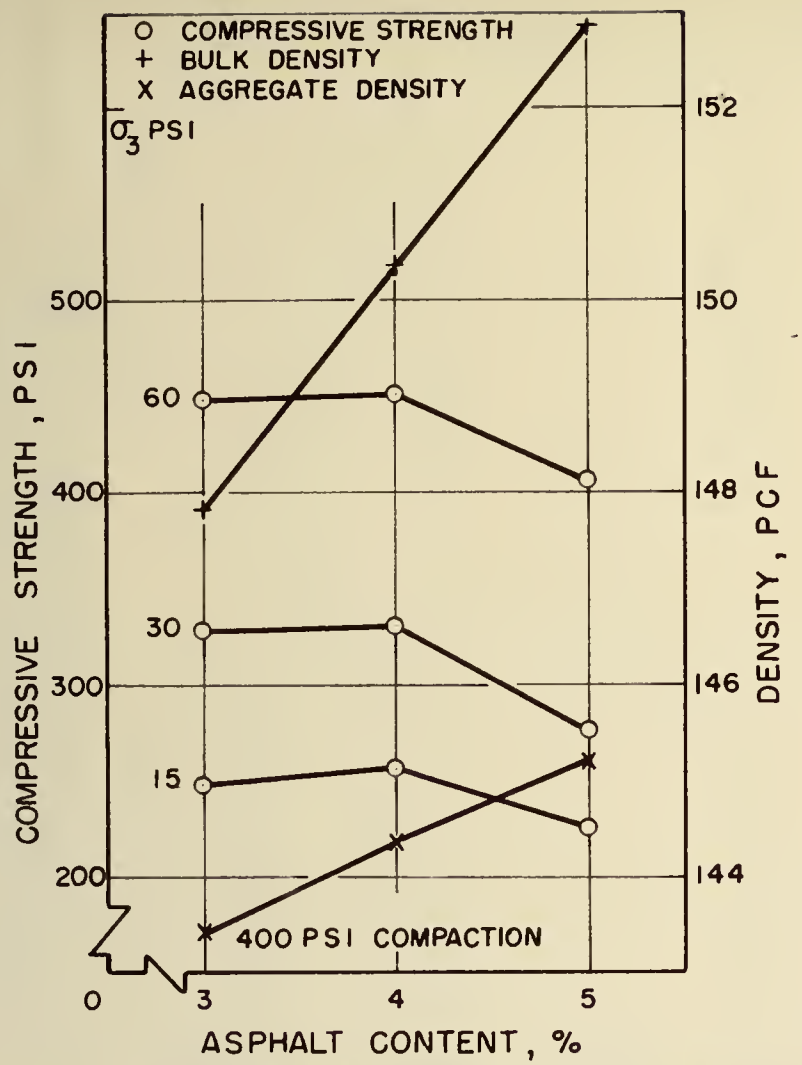


FIGURE 24. COMPRESSIVE STRENGTH AND DENSITY VS ASPHALT CONTENT – LOW COMPACTION PRESSURES



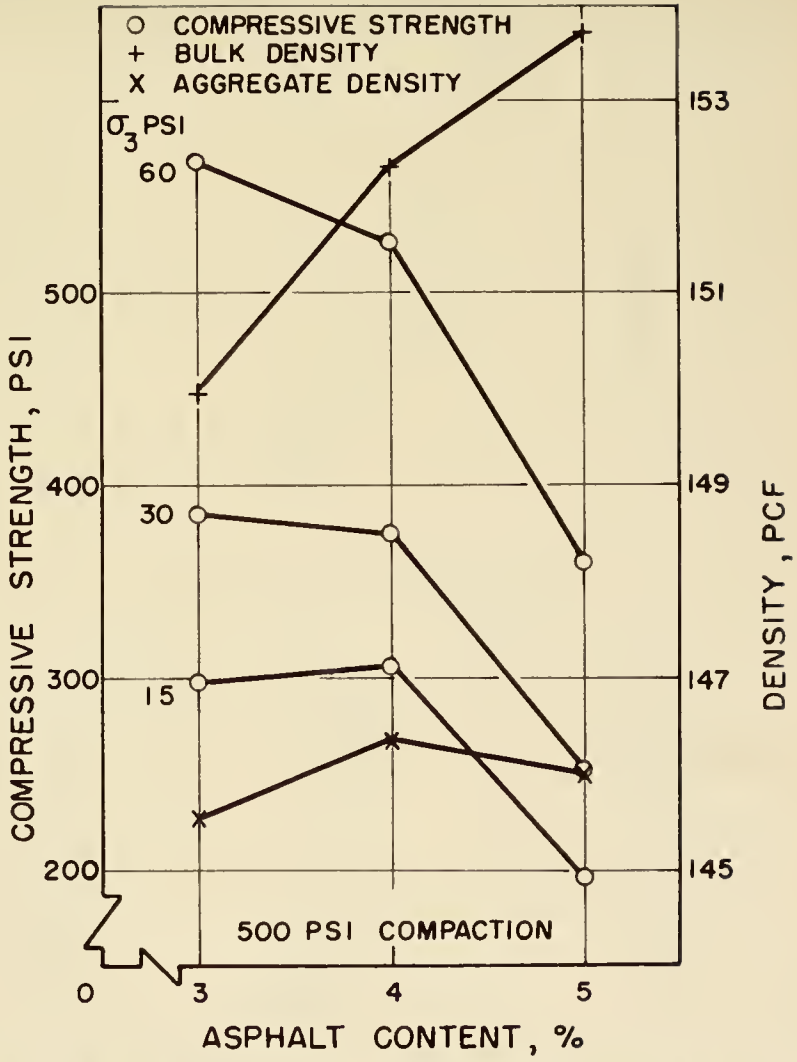
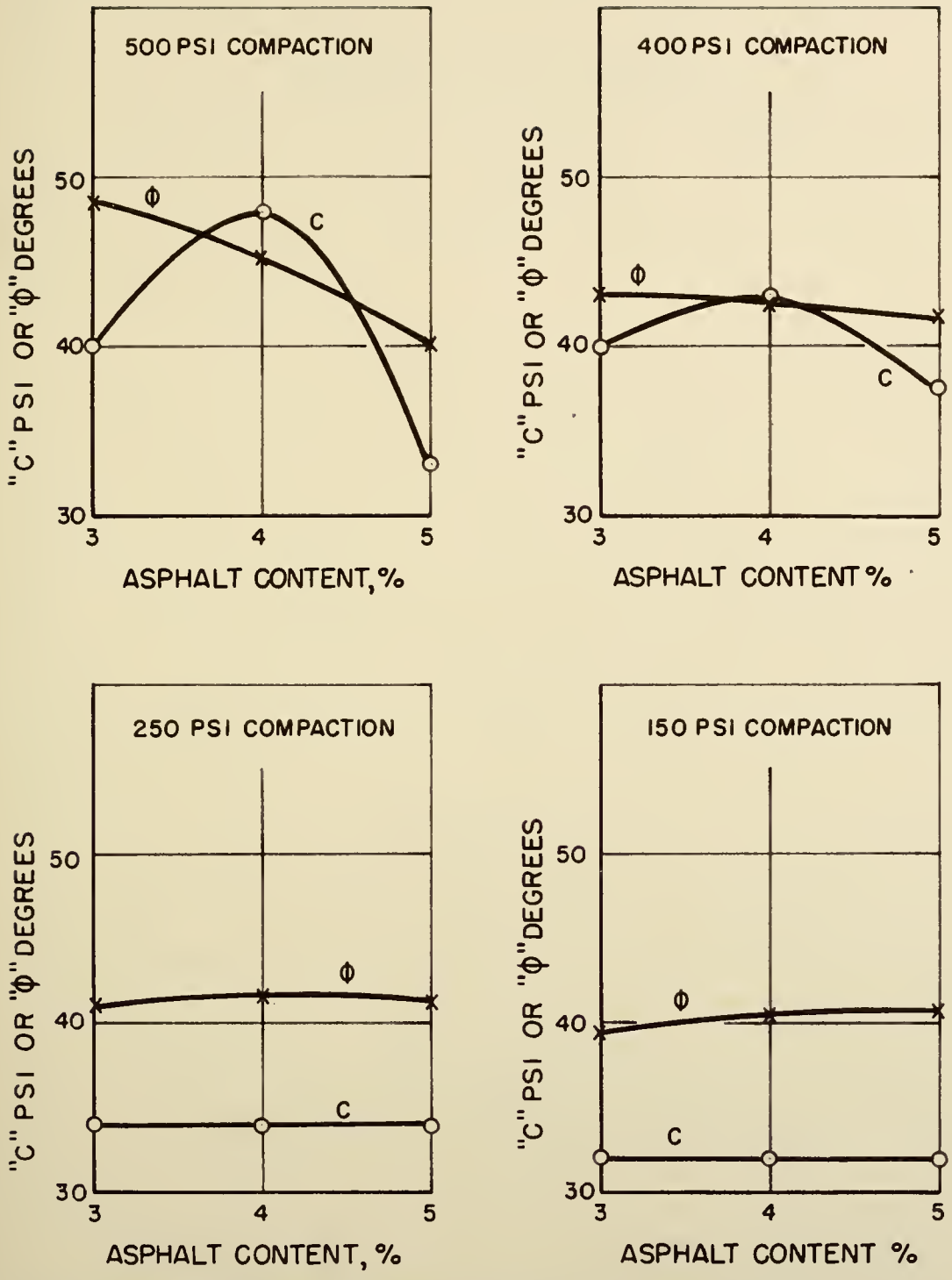


FIGURE 25. COMPRESSIVE STRENGTH AND DENSITY VS ASPHALT CONTENT – HIGH COMPACTION PRESSURES



NOTE : "C" = cohesion ; "φ" = angle of friction.

FIGURE 26. SHEAR STRENGTH PARAMETERS AT FAILURE VS ASPHALT CONTENT

was performed. The plotted data are tabulated in Table 10 (Appendix A).

Volume Change Characteristics

The measurement of the volume changes of the specimen under triaxial test conditions constituted a major purpose of this study. The equipment used for these measurements is discussed in the section on Materials and Procedures.

The computation of the volume of the specimen at any point during the progress of the test was based on observed changes in the volume of the cell liquid as recorded by the volume measuring device and the computation of the initial bulk volume of the specimen. An initial reading of the mercury level in the volume measuring tube provided the reference to which changes in this level could be referred for the computation of volume changes. Readings were obtained at 15 seconds after the application of the confining pressure and at periodic intervals during the testing.

It is assumed that the volume change recorded as the difference in the 15-second and initial readings is due to two separate effects. The first of these is the change in the volume of the apparatus due to the confining pressure. The second is the result of the membrane being forced into the surface irregularities of the specimen. Calibration values of the volume changes of the apparatus under the various confining pressures are shown in Appendix D.

The volume observed as the difference between the calibration volume value for a given confining pressure and the 15-second reading has been treated as a volume representing surface voids of the specimen. The magnitude of this value is directly related to the amount of the

confining pressure and the surface condition of the specimen. In general, the higher the lateral pressure, the greater the measured volume assumed to be surface voids.

The amount of volume change occurring under each successive load increment was computed from the difference in the volume measuring device readings before a load increment application and just prior to the application of the following increment. Therefore, any error induced in the computations due to air in the system, erroneous calibration, personal error in initial reading, etc., is contained within the 15-second reading. The result is, that while the absolute value of the computed volume changes may be slightly in error, the general pattern of these changes and the order of magnitude of the changes are thought to be correct.

The volume of the specimen and the volume of air within the specimen were computed by the algebraic summation of the initial values of these volumes and the incremental changes computed from the differences in the mercury levels between successive readings corrected for the volume displacement due to the movement of the piston into the cell.

Figures 12 and 13 show typical changes in volumes of specimens under the applied deviator stress as a function of axial strain. For ease in comparing the magnitudes of volume changes between specimens, the changes are presented as a percentage of the initial specimen volume less the volume assumed to represent surface voids.

All specimens showed a change in volume during the testing. In all cases there was a decrease in volume followed by an increase in volume. The increase in volume became linear with axial strain and continued in

such a manner to failure. In all cases there was an appreciable increase in volume at the failure condition. The slope of the straight line portion of the volume-change plot varied with confining pressure and with aggregate density. In general, the higher the aggregate density the greater the gradient and the higher the asphalt content the lower the volume-change gradient. An increase in confining pressure for tests on specimens having similar initial conditions caused a decrease in the gradient.

The volume changes occurring as a decrease in the volume of the specimen were greatest for the higher lateral pressures. The magnitudes of these changes in any case were small - generally less than one-half of one percent of the reference volume. The greatest decreases in volume occurred for specimens having the lowest aggregate density values.

Table 7 (Appendix A) shows the magnitudes of the observed volume changes at the Proportional Limit, at two percent strain, and at the minimum sample volume. The volume changes are reported as cubic centimeters of change from the initial bulk volume less the volume assumed to be surface voids. The percentage of the reference volume represented by this change is shown in the adjacent parenthesis.

Typical volume computations are presented in Appendix B.

Void Ratio and Percent Voids

In asphalt mixture terminology, the percent voids (%V) is defined as the amount of air volume expressed as a percentage of the total volume of the specimen. In soil mechanics terms, an expression for void ratio (e) is commonly used to define the ratio of volume not occupied

by aggregate solids to the volume of the aggregate solids. Both terms have been used in this study, the former because it provides an easy reference for those familiar with asphalt terminology and the latter because the aggregate solids volume provides an unchanging reference regardless of whether the specimen expands or contracts. A mathematical relationship between these two terms is developed in Appendix E.

Void ratio and percent voids were calculated utilizing the computed volume of the air and total specimen volume based on observed volume changes and on the known fixed volume of bitumen and aggregate solids in the specimen as computed from the initial measurements of each specimen. Aggregate volumes were computed using the average bulk specific gravity of the aggregate blend and the asphalt volume was determined from the weight of asphalt in the specimen less the weight absorbed by the aggregate. Sample computations of void ratio and percent voids are shown in Appendix B while a summary of these values at the initial condition, the Proportional Limit, and two percent strain are presented in Table 7 (Appendix A).

The change in void ratio and in percent voids during a test follows the same general pattern as that of the total volume change. Initially there is a decrease in these values and then an increase to failure. Both terms show a very gradual change in value under the intermediate applications of load. Changes are greatest under the initially applied loads and as the specimen approaches failure. Figure 27 shows the void ratio and percent voids plotted against total applied axial stress for a typical test series.

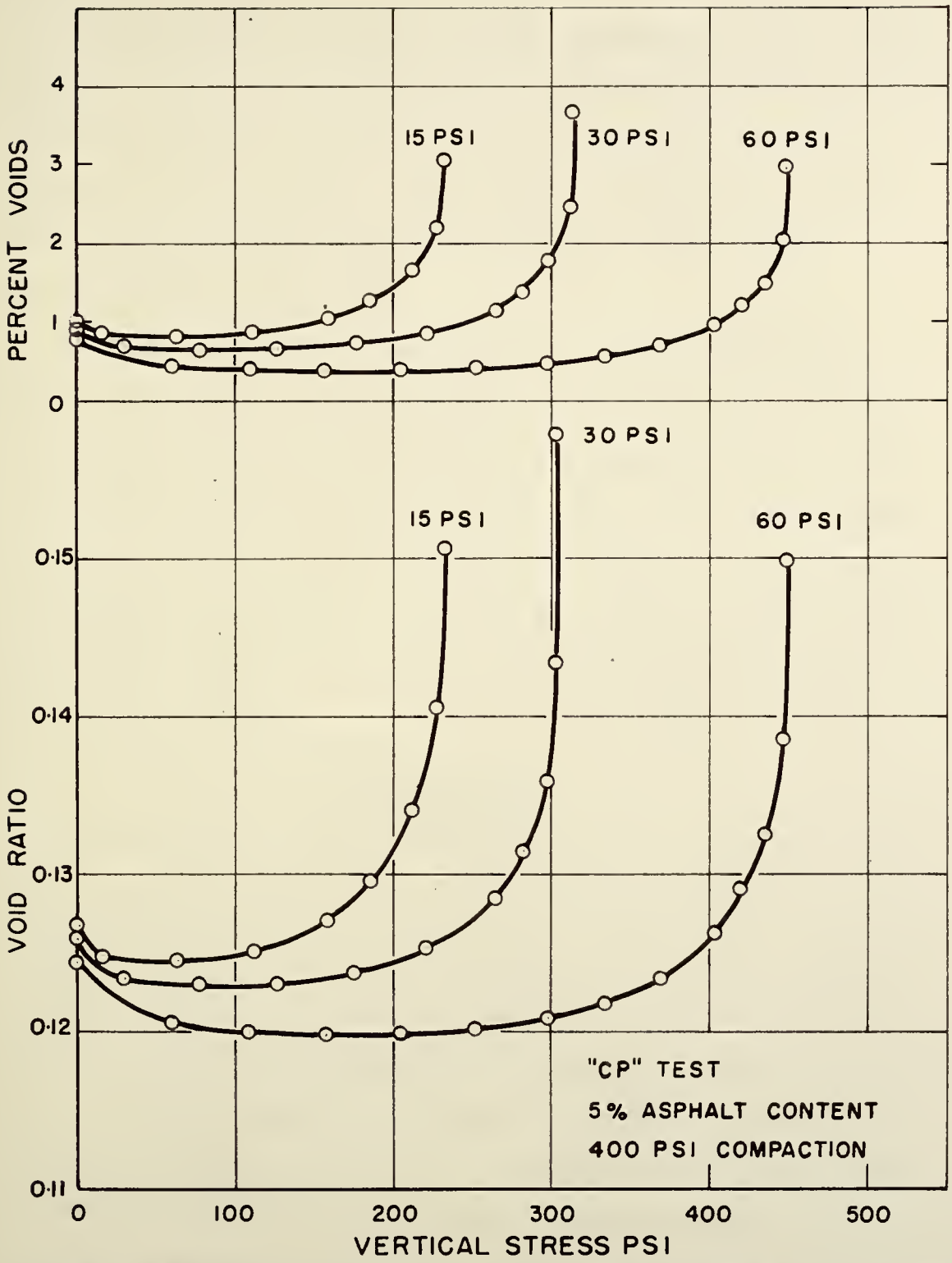


FIGURE 27. PERCENT VOIDS AND VOID RATIO VS STRESS

Figures 28, 29, 30, and 31 show the relationship between deviator stress and void ratio and percent voids at the Proportional Limit and at two percent strain for all routine tests performed on specimens prepared at 4 and 5 percent asphalt content and 150 and 400 psi compaction pressure. Similar data are tabulated in Table 7 (Appendix A) for these and all other tests.

Figures 32 and 33 show like data in a slightly different form. These figures present a plot, for all routine test results, of the deviator stress versus the percent voids at two percent strain. The curves that are approximately vertical represent the deviator stress variation with percent voids at two percent strain for a given confining pressure and compaction pressure. The percent voids data shown in Figures 29 and 31 for the two percent strain condition have been plotted also in these figures.

Deformation Ratio

The Deformation Ratio is a defined term that is applied to the ratio of lateral strain to axial strain for a material that is not subject to the strict application of Poisson's Ratio. Poisson's Ratio is defined as the ratio of lateral strain to axial strain within the elastic limit of the material concerned. As asphaltic mixtures are not strictly elastic throughout any range of deformations and as it is desirable to know the ratio of strains throughout the entire range from zero to the failure condition, the term Deformation Ratio is introduced. It should be noted that this term is analogous to Poisson's Ratio and is defined merely to extend the range of the concept expressed by Poisson's Ratio for elastic materials.

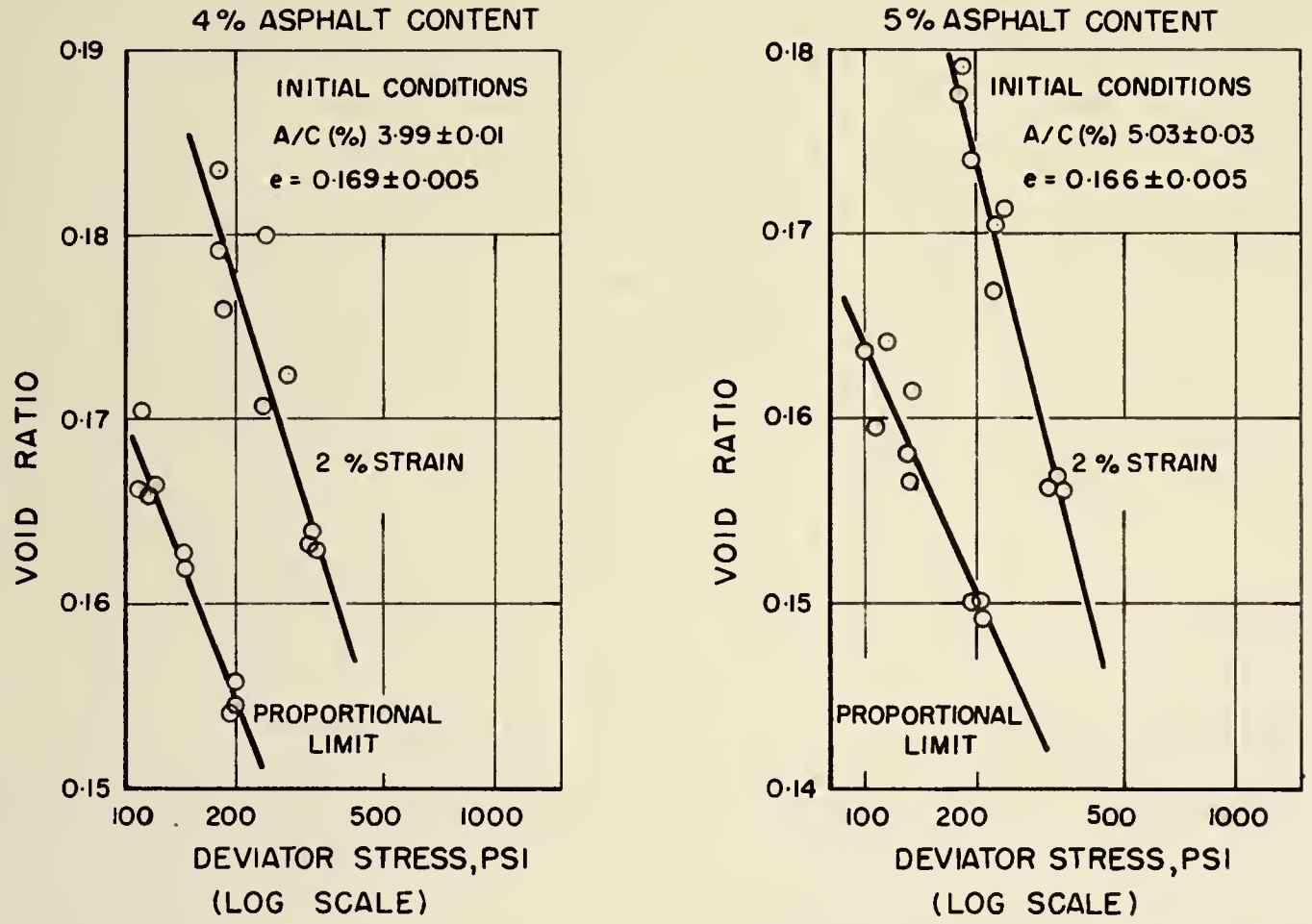


FIGURE 28. VOID RATIO VS DEVIATOR STRESS–150 PSI COMPACTION

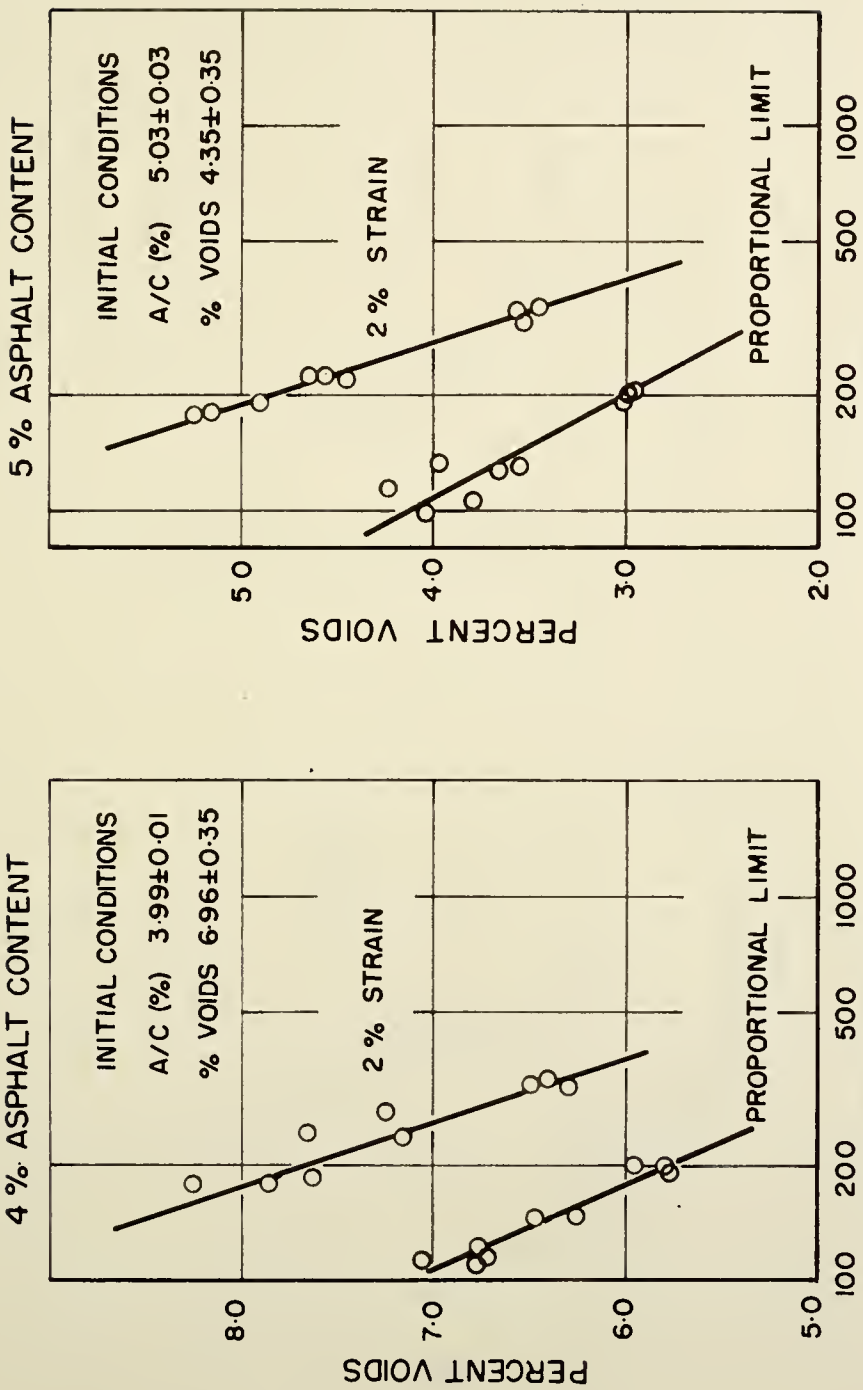


FIGURE 29. PERCENT VOIDS VS DEVIATOR STRESS–150 PSI COMPACTION

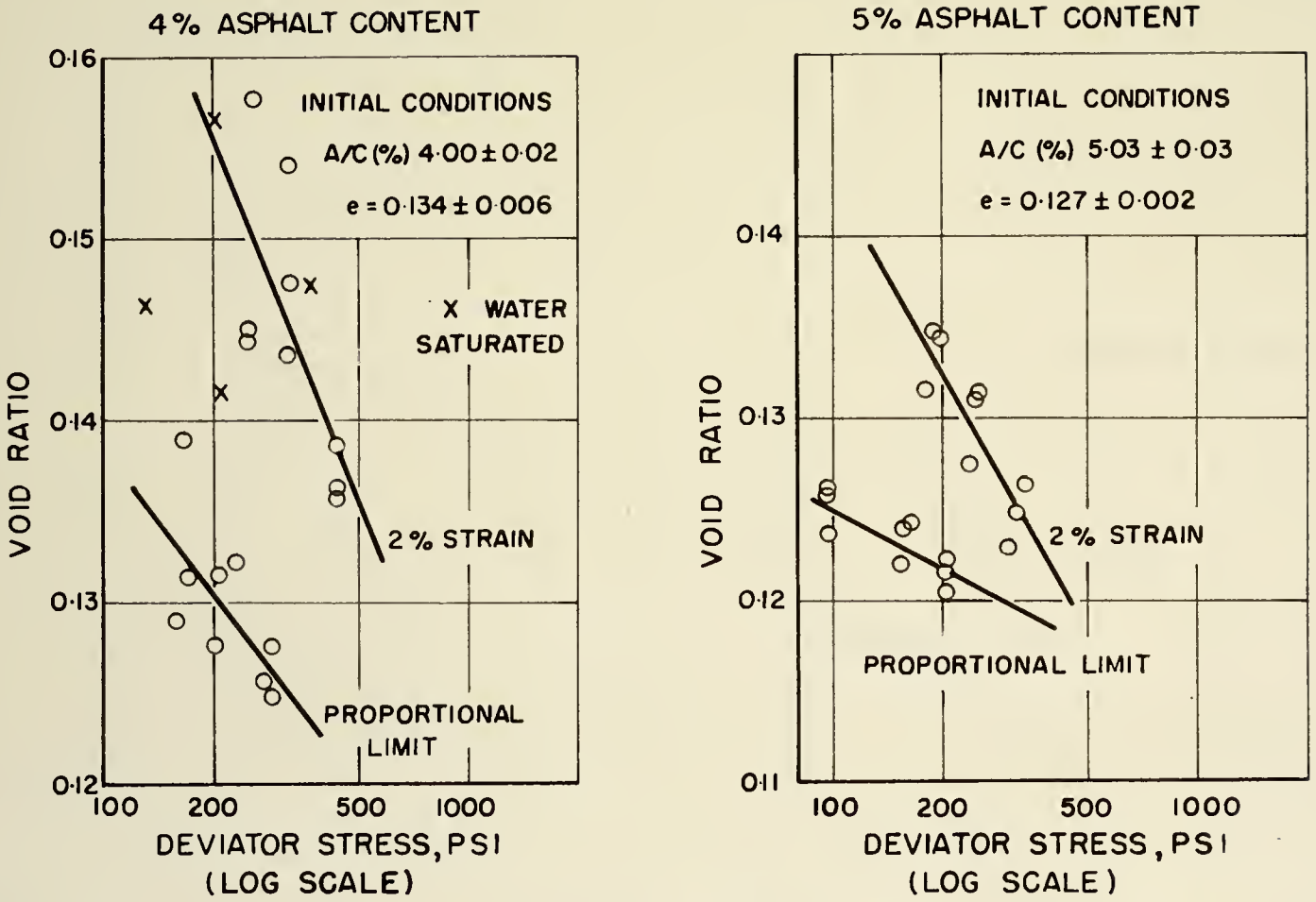


FIGURE 30. VOID RATIO VS DEVIATOR STRESS – 400 PSI COMPACTION

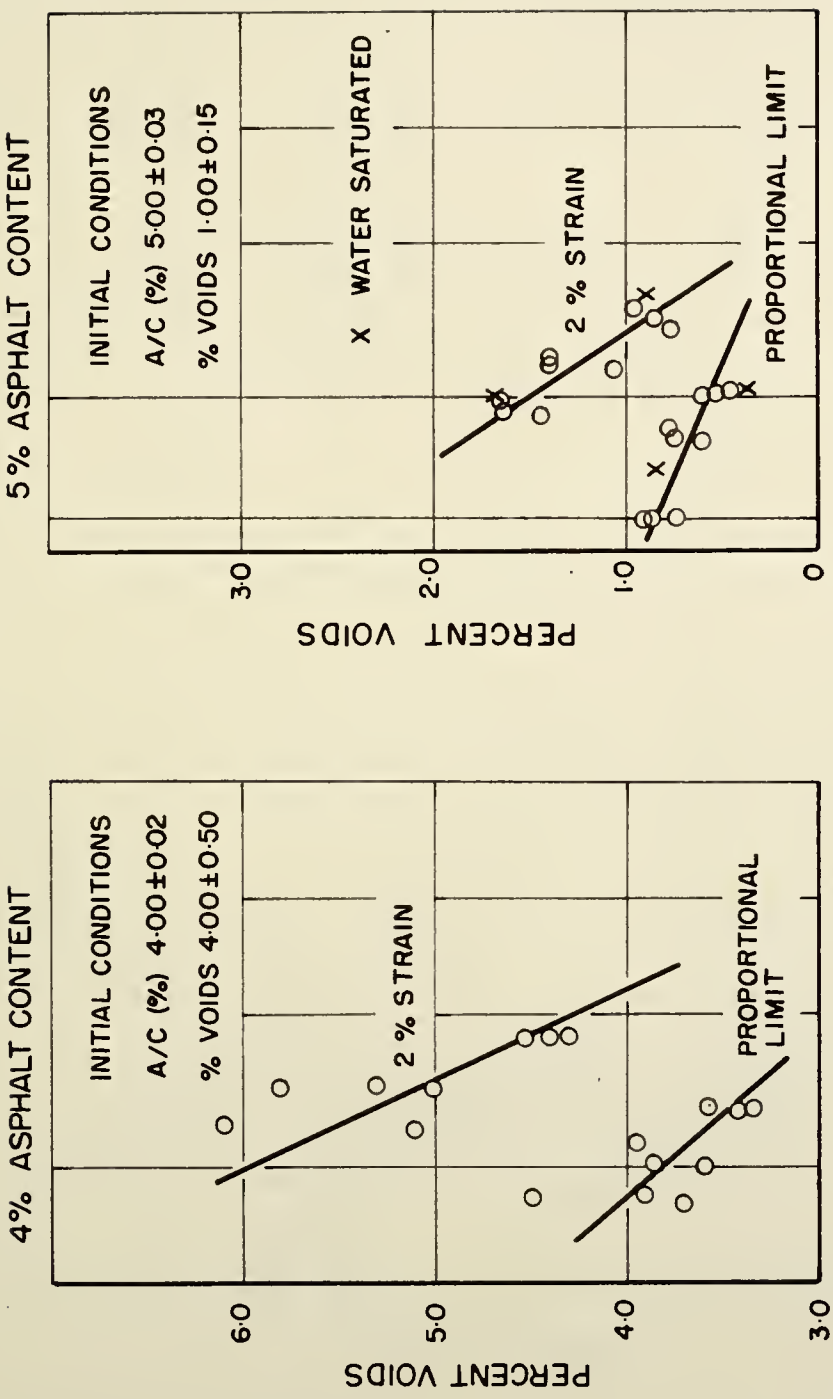


FIGURE 31. PERCENT VOIDS VS DEVIATOR STRESS — 400 PSI COMPACTION

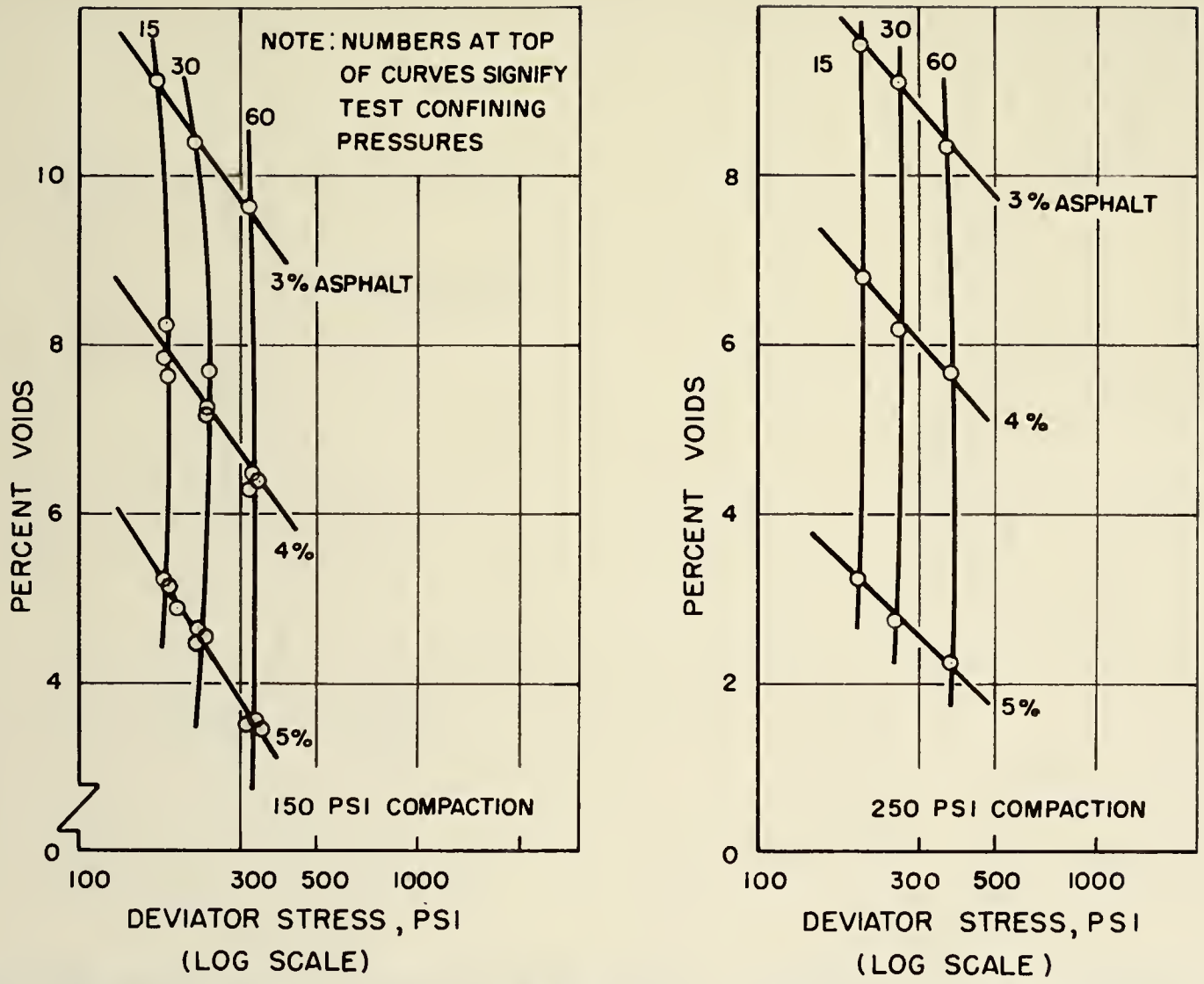
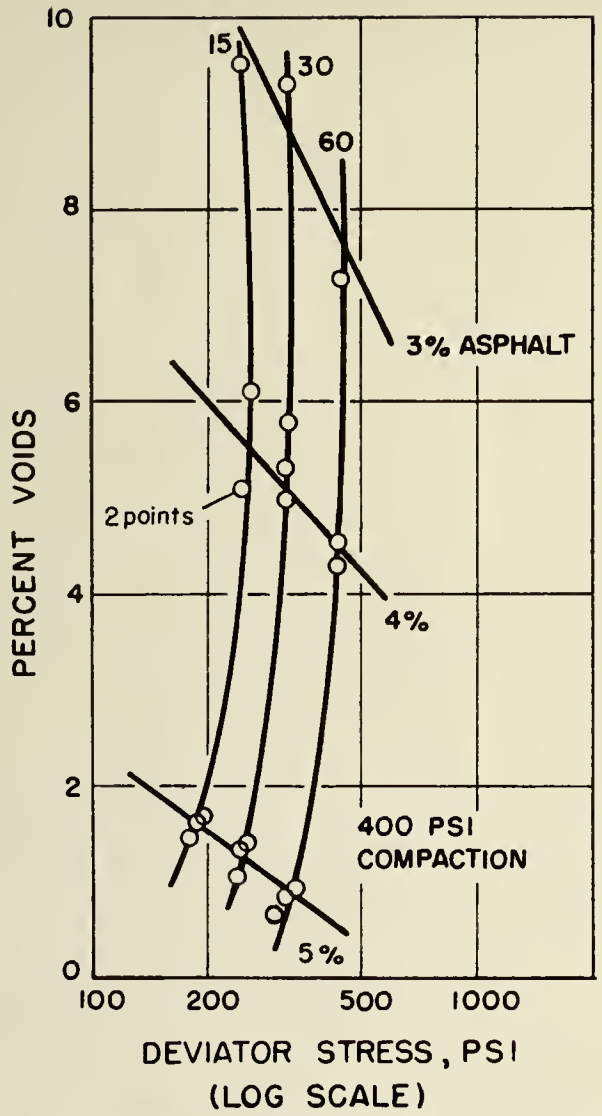


FIGURE 32. PERCENT VOIDS VS DEVIATOR STRESS AT 2% STRAIN — LOW COMPACTION PRESSURES



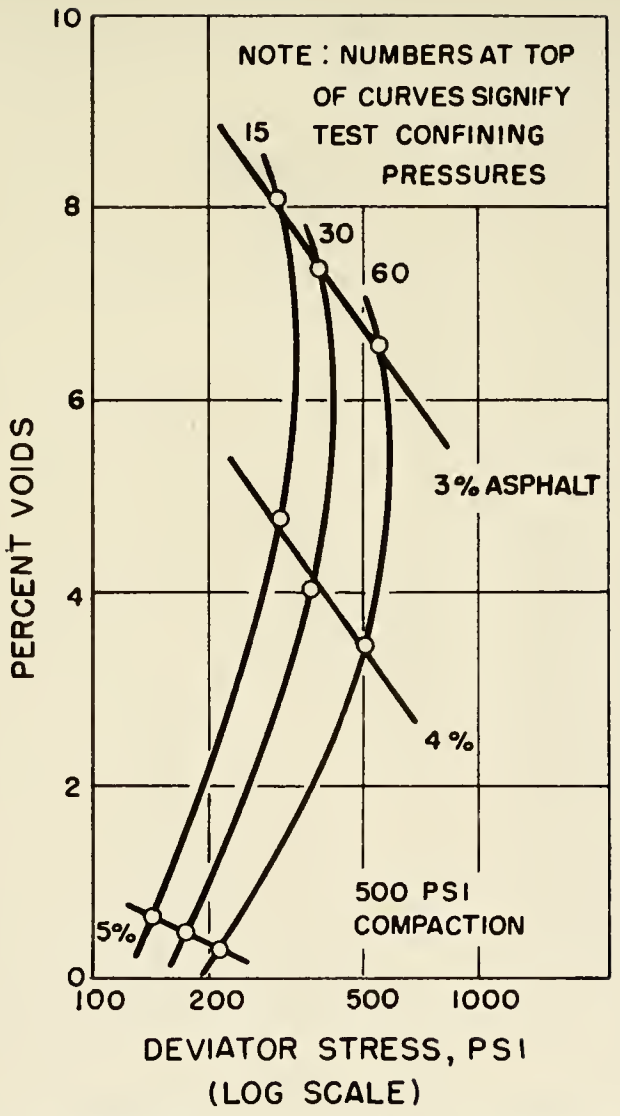


FIGURE 33. PERCENT VOIDS VS DEVIATOR STRESS AT 2% STRAIN – HIGH COMPACTION PRESSURES

Data observed during the testing permit a computation of both the axial and volumetric strains that occurred under any given load application. In the development used, these strains were referred to the specimen height and volume existing just prior to the application of the first deviator stress increment.

If it is assumed that the volume changes that occurred are uniform within the cylindrical specimen, then it is possible to develop a relationship defining the Deformation Ratio (D_r) in terms of volumetric strain and axial strain. This relationship may be stated $D_r = -(\epsilon_v - \epsilon_a)/2\epsilon_a$, where ϵ_v and ϵ_a are volumetric and axial strain respectively. The mathematics of this development are presented in Appendix E.

The D_r values have been computed for four series of "Q" tests, each at a different combination of asphalt content and compaction foot pressure. The results of these computations are presented graphically in Figure 34 for one series and in tabular form in Tables 11 and 12 (Appendix A) for the four test series investigated. In general, the presentation follows a scheme suggested by Jakobson (69).

The figure showing the results of the D_r computations presents them as plots of D_r versus $\sin \phi_1$, where ϕ_1 is the angle of shearing resistance that has been mobilized at that particular point in the test. The values of $\sin \phi_1$ were computed utilizing the fundamental relationship $\sigma_1 = \sigma_3 \tan^2(45 + \phi/2) + 2c \tan(45 + \phi/2)$ for a material possessing both friction and cohesion. To facilitate the computation it was assumed that the term $2c \tan(45 + \phi/2)$, evaluated at the failure condition, was a constant and was acting at all stages of the test. This term was

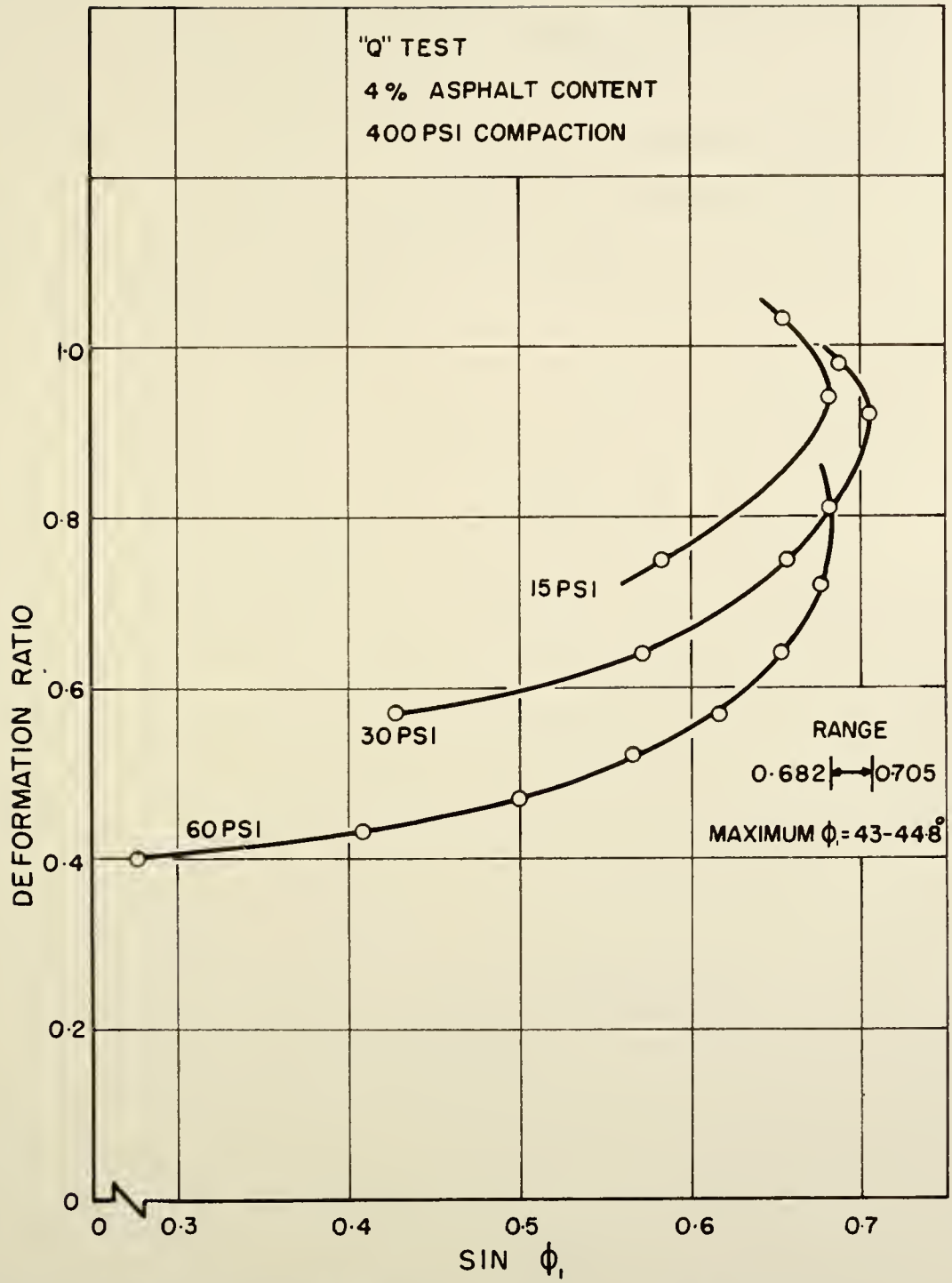


FIGURE 34. DEFORMATION RATIO VS ANGLE OF SHEARING RESISTANCE

subtracted from the major principal stress acting at each point and sin ϕ_1 computed using the frictional resistance values only. An illustration of this computation is presented in Appendix B.

Appendix B also presents a typical set of computations leading to the data required for a plot of D_r versus sin ϕ_1 for a given test. The particular computation presented is shown graphically as the σ_3 = 30 psi curve in Figure 34.

DISCUSSION OF RESULTS

The discussion of the results obtained in this study is sub-divided into eight sections. Five of these are discussions of the results obtained which have direct bearing on the purposes of the investigation, while the remaining three refer to the results that developed as an incidental part of the investigation but which are of interest in the study of bituminous mixtures.

Equipment

Two major pieces of equipment, peculiar to this study, merit discussion. The controlled temperature room was purchased in order to provide a constant temperature atmosphere in which to perform all tests. The volume measuring device was constructed to provide a means for establishing the volume changes of a specimen subjected to triaxial compression if such changes occurred.

The controlled-temperature room performed well during the entire period of usage. An established temperature of 25°C (77°F) was maintained at the sensing element within the room and in the water used for a cell liquid. Some variation in temperature was noted within the room but, in general, this variation was less than $\pm$ 0.5°C from the average air temperature at the point of measurement. The temperature records show a daily 25°C trace with a variation of the order of $\pm$ 0.5°C. The exact temperature was not so critical a part of this study as was the

constancy of the temperature during testing. The uniformity of temperature was adequate for test purposes.

Some disadvantages are inherent in the controlled temperature room. The room, as constructed, provides for a recirculation of air within the room. As a result, the room air becomes stale and odorous after a long period of use. The size of the room limits the number of persons that can work within it when the triaxial equipment is in place.

The volume measuring device performed well throughout the period of testing. Initially there was some difficulty in removing all entrapped air from the system, but this was overcome by patient bleeding and refilling of all lines and containers. The use of an epoxy resin seal on all plastic tubing joints was completely effective in eliminating leakage from these joints under the range of confining pressures developed during the testing.

Four calibration tests were performed at various times during the period of testing. The calibration test results are presented graphically in Figure 35 (Appendix D). Care was required in performing the calibration tests. Occasionally repeated applications of a confining pressure on a given calibration test set-up yielded different values. This was assumed to be due to air in the system, a poor initial reading, or other operator error. In such cases, the calibration runs were repeated until consistent results were obtained.

In summary, it may be stated that the volume measuring device performed well for the measurement of differences in volume under successive loads but some difficulty was experienced in establishing the initial volume change under the application of the lateral pressure. As all

computations were based on differences in readings of the volume measuring device, it is felt that the results obtained using the apparatus are correct as to trend and order of magnitude but may be slightly in error as to the exact value as a result of the initial reading and calibration difficulties.

Specimen Fabrication

The technique developed for the fabrication of four-inch diameter by eight-inch high specimens was very satisfactory. Initial trials using this procedure on approximately fifty specimens indicated that the variation in density throughout a specimen was generally less than 0.5 percent of the average density of the cut sections. Asphalt contents were found to vary less than 0.3 percent from the average of the asphalt contents of the sections.

Some difficulties were encountered in using the kneading compactor at low foot pressures. Feeding the material into the mold during compaction leads to a drifting of the applied air pressure from the preset amount. The result was that a continual adjustment of the air pressure was necessary. At high foot pressures a similar phenomena was noted though to a lesser extent.

It was obvious from observations during compaction and by inspection of the compacted specimens that some aggregate breakdown occurred during the compaction process. The extent of this breakdown was not investigated.

During the early stages of the testing program, it was noted that uniformity of density between the three specimens prepared at one time was not always as good as desired. As a result, four specimens were

made for the various combinations of asphalt content and compaction pressure during the latter period of testing. The three specimens having the best uniformity of density were selected for testing while the fourth was held in reserve for a re-test that might be necessitated by membrane failure, equipment leakage, etc.

Effect of Test Procedure on Observed Strength

The first routine test series were performed with specimens prepared at 4 and 5 percent asphalt content using compaction pressures of 150 and 400 psi. Tests were performed on similar specimens using three different conditions of drainage. Quick or "Q" tests were performed in which the specimen drainage valve was closed during the application of all load increments. Consolidated-Quick or "Qc" tests were accomplished in which the drainage valve was open during a one-hour period of confining pressure application and then closed for the application of all axial load increments. The Consolidated-Partially Drained or "CP" tests were completed in a fashion similar to the "Qc" tests except that the drainage valve was kept open during the application of all loads.

In no case was there sufficient difference between the results of "Q", "Qc", and "CP" tests for specimens prepared at the same asphalt content and compaction pressure to be noticeable (see Figures 16-21 and Table 7, Appendix A). Variations in deviator stress of approximately 10 percent were observed but these did not follow a pattern that could be satisfactorily correlated with drainage conditions. It was assumed, therefore, that these differences were random variations due to non-uniformity in the test specimens or to other uncontrolled test errors.

The effect of drainage was found to be equally unnoticeable for tests performed on three specimens using a slow rate of loading (see tests 63-65, Table 7, Appendix A).

The results of the first thirty-nine tests indicated little or no difference in the strength or stability of similar specimens tested under different conditions of drainage. The remainder of the tests were performed, therefore, using the "Q" test procedure only.

Evidence is plentiful to indicate that negative pore pressures do develop within the specimens during testing. An expansion of the specimen under conditions of no drainage implies an increase in the volume of air within the sample and, therefore, a negative pore pressure. It was noted also that if the drainage valve was opened at the completion of a "Q" or "Qc" test there was an obvious sound of moving air. That this air was moving into the specimen was evidenced by a suction developed on the operator's hand if held over the drainage outlet.

It must be concluded, in light of evidence of negative pore pressure and uniformity of observed strength results regardless of drainage conditions, that, though pore pressures apparently develop, their effect was so small compared to the strength of the mixture that they did not influence the test results. This conclusion appears to be the only rational one possible for loading times of up to one hour between load increments.

Volume Changes

In every specimen tested, regardless of test procedure or initial conditions, a change in volume of the specimen was noted. All tests followed the same general pattern of volume change. Typical curves of

volume change with axial strain are shown in Figures 12 and 13. The specimen decreased in volume under the application of the confining pressure with the magnitude of the decrease a function of the magnitude of the confining pressure all other things being equal. Further, but smaller, decreases in volume occurred under each of the first few axial load increments. Additional axial loading produced a volume increase that continued to failure. The magnitude of the volume changes that occurred at various points throughout a test are tabulated in Table 7 (Appendix A).

The amount of the reduction in specimen volumes was small. In nearly every case the volume decrease amounted to less than one-half of one percent of the specimen volume less the volume of surface voids. In all cases, the axial strain at the point of minimum specimen volume was less than that at the Proportional Limit. In nearly all cases, the volume change increased linearily with axial strain over the range of strain values from slightly less than those at the Proportional Limit to values at failure.

The slight reduction in specimen volume noted during the early portion of all tests was thought to be due to several causes. A portion of the reduction was possibly due to the recovery of expansion that may have occurred upon release of compaction pressures applied during specimen fabrication. Another portion of the reduction may have been due to elastic compression of specimens under the confining pressures. In addition, some decrease in volume may have occurred as a result of plastic deformation of the bitumen.

It seems apparent that dilatancy occurs with bituminous mixtures subjected to triaxial shear. This can be explained in a manner similar to that utilized for explaining the same phenomena in dense-granular soils. As shearing begins to occur, grains are forced to move apart in order to ride over one another. This expansion continues until all particles on the shearing plane are free to move. At this point the maximum strength has been reached. The dilatancy or volume increase with axial strain occurs at a uniform rate during shearing and its value is a function of the initial density of the aggregate mass.

A study of the volume change gradients developed between points corresponding to zero percent volume change and two percent axial strain (see Table 7, Appendix A) showed the following effects for the bituminous mixtures studied: The magnitude of dilatancy increased with increasing compaction pressure regardless of asphalt content. At a constant compaction pressure it was observed that the rate of volume increase with axial strain was greatest for the lower asphalt contents. The former result is as expected based on experience with dense-graded granular soils. The latter observation is not explained so readily. It appears probable that the magnitude of dilatancy is less for the higher asphalt contents because of the presence of thicker films of the viscous binder between particles. This would permit the specimen to accept greater axial strains with less movement of the aggregate particles than if the asphalt film were of less thickness as is the case for the lower asphalt contents. The asphalt would permit plastic deformation of the mixture and would reduce the movement necessary for particles to move over one another.

Relationships Between Void Content and Strength

Computations of the void ratio and percent voids at two percent strain and the corresponding deviator stress show that there is a definite relationship between voids and strength for the mixtures and conditions studied. Figures 28, 29, 30, and 31 illustrate this relationship for both void ratio and percent voids for tests performed at all conditions of drainage on specimens prepared at 4 and 5 percent asphalt content and 150 and 400 psi compaction pressure.

With limited deviation, the relationship of void ratio and percent voids, both at two percent strain and the Proportional Limit, is linear with the logarithm of the deviator stress. This linear relationship holds for all tests on specimens prepared by the same compaction pressure and at the same asphalt content regardless of confining pressures or drainage conditions during the test. The two percent strain curves indicate that the linear relationship appears to be valid for cases where the values of percent voids and void ratio are both larger and smaller than the initial values of the void ratio and percent voids. Indications are that the slope of the straight line is larger for specimens of the lower asphalt content at any given compaction pressure than for specimens of the higher asphalt content and the difference between slopes for two asphalt contents becomes less at the lower compaction pressure.

In order to verify the uniqueness of the established relationship, two samples were prepared at 4 percent asphalt content and 400 psi compaction pressure and water saturated prior to testing. The strength-voids relationships of specimens fabricated at 4 percent asphalt content

and 400 psi compaction pressure are expressed by those of Figures 30 and 31. The water saturation served to reduce the amount of air in the specimens to a quantity essentially equal to that in specimens prepared by the same compaction pressure but at 5 percent asphalt content. Thus the void ratio values of the water-saturated specimens remained nearly equal to those of similar specimens without water saturation, while the percent voids values were reduced as though the asphalt content had been increased. Referring to Figure 31, it is noted that the percent voids -- strength relationship for the water-saturated specimens fits the relationships for 5 percent asphalt content very well for both two percent strain and Proportional Limit cases. Figure 30 shows that there is reasonable correlation of void ratio and strength at two percent strain for the water-saturated specimens with that of similar but unsaturated specimens. The relationship of void ratio and strength at the Proportional Limit for the water-saturated specimens shows some deviation from the established curve.

The deviation of the void ratio data for the water-saturated specimens from the curve established in Figure 30 is probably due to the increase in void ratio resulting from the slight increase in specimen volume as a result of water saturation. This increase in volume was considered in the computations of the water-saturated specimens test data, but the effect was to change the initial void ratio conditions slightly from those of specimens not saturated.

The diagonal straight lines of Figures 32 and 33 show the same relationship between percent voids and deviator stress at two percent strain as shown in Figures 29 and 31 and also present similar data for

routine tests at other combinations of asphalt content and compaction pressure. In addition, Figures 32 and 33 show that apparently there is a unique relationship between percent voids and deviator stress at two percent strain for a given confining pressure and compaction pressure regardless of the asphalt content at which the specimens were prepared.

A study of Figure 33 suggests the possibility of a critical value, with respect to strength, of the percent voids at two percent strain. In particular, the curves for 500 psi compaction are sufficiently curved to indicate the likelihood of a peak value of deviator stress occurring between approximately four and six percent voids. As compaction pressure was decreased the curvature of the plot of percent voids versus deviator stress at a particular confining pressure decreased until a linear relationship is indicated for the 150 psi condition. Unfortunately, there are insufficient data to clearly define the concept of a critical percent voids with respect to strength.

The implication of Figures 32 and 33 is that a family of curves may be developed for a given aggregate, a given asphalt, and an established compaction and triaxial test procedure that will define the deviator stress and percent voids at two percent axial strain. As there is a mathematical relationship between percent voids and void ratio, a similar set of curves could be developed using void ratio in lieu of percent voids.

It should be noted that the above results and conclusions are applicable only to the aggregate, aggregate gradation, and penetration grade asphalt cement used in this study as well as for the particular compaction and testing procedures used. It is felt, however, that

similar results for other aggregates, aggregate gradations, and asphalt cements are probable.

Variation of Strength With Asphalt Content

In the interest of expanding the general knowledge of the relationship between asphalt content and compressive strength under conditions of triaxial compression testing, data from all routine tests are presented in a plot between these variables in Figures 24 and 25. Superimposed over the strength curves in these figures are curves expressing the relationship between aggregate and bulk densities and asphalt content.

From Figure 24 it may be noted that the variation in strength with asphalt content at any given confining pressure is quite low for the 150 and 250 psi compaction pressures. In light of the variation in bulk density and aggregate density for the same conditions, this figure provides further verification of the lack of correlation between specimen density and strength.

Figure 25 presents similar data for 400 and 500 psi compaction pressures. The variation in strength with asphalt content at any given confining pressure is noticeable in this figure. Essentially equal strength values were developed under the 400 psi compaction pressure at 3 and 4 percent asphalt contents, but there was a rather sharp decrease in strength at the 5 percent asphalt content condition. This decrease is pronounced even though bulk density and aggregate density show a continuing increase with asphalt content. The 500 psi compaction results again show about the same strength values for the two lower confining pressures; however, a strength decrease is noticeable from 3 to 4 percent asphalt content for the 60 psi confining pressure. It is apparent also

that the sharp decrease in strength at the 5 percent asphalt content is reflected by a decrease in aggregate density even though bulk density continued to increase.

On the basis of Figures 24 and 25 it is apparent that compaction pressure greatly affects the observed compressive strength of a specimen formed from a given mixture. It is apparent also that a density criterion for suitability of a mixture from a strength standpoint is not reliable. Furthermore, it appears that the aggregate density of a specimen is a more significant measure of the change of strength than is bulk density.

The apparent shear strength parameters, "c" and "$\emptyset$", were established by a Mohr's circle of stress analysis for the failure conditions and for the stress at two percent axial strain for all test series. Typical Mohr circles for each of these strength criteria are shown in Figures 22 and 23. In all cases the envelope of stress was fitted to the circles of stress as a straight line. In some cases the tangency of the circles to the fitted straight line was not perfect (see Figure 23). The variation from a straight line envelope, however, was so slight that a curved envelope did not appear to be justified. It is conceivable that further tests at higher confining pressures would emphasize the possibility of a curved envelope of failure.

Figure 26 and Table 10 (Appendix A) present the results of a Mohr's circle analysis of the shear strength parameters at the failure condition for all routine tests. It may be readily observed that there is a decided decrease in the value of the observed angle of internal friction with increased asphalt content at the two higher compaction pressures.

This decrease is as expected when it is realized that asphalt film thickness increases with asphalt content and thus tends to reduce the intimate contact between aggregate particles. At the two lower compaction pressures there is little change in the observed angle of internal friction with changes in asphalt content. In fact there appears to be a slight increase in this parameter as asphalt content increases. It is thought that the aggregate density under these compaction pressures is such that an increase in asphalt content serves to improve compaction efficiency and that a sharp decrease in observed angle of internal friction would occur at still higher asphalt contents where these low compaction pressures are used.

The cohesion parameter relationships shown in Figure 26 follow an interesting pattern. The following comments represent an attempt to explain these relationships utilizing the concepts advanced by other investigators. As an example, see Herrin (70). As compaction pressure increases the variation of cohesion with asphalt content varies gradually from essentially no change to a curve that is markedly convex. For a given asphalt content the value of cohesion increases with compaction pressure for 3 and 4 percent asphalt contents but the cohesion at 5 percent asphalt content decreases upon an increase in compaction pressure from 400 to 500 psi compaction. Apparently the increase in cohesion that occurs with an increase in compaction pressure results from the higher compaction pressure forcing the coated particles into more intimate contact and increasing the contact area between coated particles. If the density of the specimen is increased by increased compaction pressures to such a point that the asphalt tends to develop thicher films between particles, the particles are forced apart and the contact area

between the coated particles is reduced. This is apparently the case for the change of compaction pressure from 400 to 500 psi for 5 percent asphalt content. The result is a reduction in cohesion.

For a given compaction pressure an increase in asphalt content acts to increase lubrication and to facilitate compaction. This in turn increases the contact area between coated particles and in so doing increases the value of cohesion to a maximum. Beyond the critical value of asphalt content, additional bitumen tends to form thicker films on the particles and to force them apart. The result is a decrease in contact between the coated particles and in the value of cohesion. The foregoing explanation of the effect of asphalt content on cohesion at a given compaction pressure is clearly illustrated at 400 and 500 psi compaction pressures in Figure 26. The explanation, however, does not appear to apply to the relationship exhibited by the two lower compaction pressures. It is thought that further increases in asphalt content beyond the quantities used in this study at the low compaction pressures would result in a critical value of asphalt content and a decrease in cohesion for asphalt contents beyond the critical value.

It is quite likely that apparent cohesion is not the proper parameter to use for expressing the relationships between cohesion, asphalt content, and compaction pressure. Future studies may permit the establishment of the Hvorslev true cohesion parameter and this concept possibly will provide further understanding of these relationships.

Modulus of Deformation

Figure 15 and Tables 8 and 9 (Appendix A) present the results of the computation of the Modulus of Deformation for all tests performed. The term Modulus of Deformation (C_d) has been used to define the ratio of the deviator stress to axial strain at a point in keeping with the

established terminology of flexible pavement design. For this study, the point at which the Modulus of Deformation is defined is the Proportional Limit.

These data are included primarily to provide further information on the order of magnitude of the Modulus of Deformation for bituminous mixtures and to indicate, in general, the trend in the variation of this measure of strength with asphalt content and compaction pressure.

From Figure 15 it is interesting to note that at the low compaction pressure there is very little difference in the measured Modulus of Deformation for 4 and 5 percent asphalt contents. The values obtained were so similar that a single curve is fitted to the data for both asphalt contents. However, there is a marked difference in the Modulus of Deformation for the two asphalt contents at the higher compaction pressure. It would appear that the Modulus of Deformation is primarily a function of the aggregate density at the lower compaction pressure. At the higher compaction pressure both the strength and strain at the Proportional Limit were affected by the asphalt content in such a manner as to yield much lower values of the Modulus of Deformation for the higher asphalt content. It is quite likely that the results at the higher compaction pressure are due to the thickness of the asphalt film between particles -- the greater the film thickness the less the Modulus of Deformation.

Deformation Ratio

Little or no information is available in the literature on the value of Poisson's Ratio for bituminous mixtures resulting from direct measurement. With the realization that data of the type gathered in

this study could provide some information on the subject, a limited investigation was initiated. The strict definition of Poisson's Ratio is not applicable to non-elastic materials; therefore, the term Deformation Ratio was defined in an analogous fashion to Poisson's Ratio for the purpose of extending the scope of the Poisson's Ratio concept.

Figure 34, Tables 11 and 12 (Appendix A) and Appendix B present the results and computations used for the investigation. The approach to the study is based on a scheme suggested by Jakobson (69) in a study of the Poisson's Ratio of sands.

It is thought that the developed Deformation Ratio data has two significant uses. Primarily, the data are intended to show the order of magnitude of the ratio of the average lateral strain to the axial strain that might be expected during the compression of a dense-graded bituminous mixture. Secondly, the data show the need for strain to develop in order to develop frictional shearing resistance.

In all cases studied it is apparent that the angle of shearing resistance is mobilized gradually with a continuing increase in the Deformation Ratio. This increase is continued to a point at which maximum frictional resistance is developed. With further deformation the frictional resistance tends to decrease or, at best, the frictional resistance remains constant.

The values of the angle of shearing resistance at which the maximum frictional shearing resistance is mobilized should represent the apparent angle of internal friction of the material at failure. In the four cases investigated, the range of the maximum value of the angle of shearing resistance for the three tests composing each case checked

very well with the apparent angle of internal friction established for each case by the Mohr circle of failure analysis. (See Figures 34 and 26 and Tables 11 and 12, Appendix A).

SUMMARY OF RESULTS

The following comments serve to recapitulate the findings of this study.

1. The controlled temperature room used to maintain a constant temperature of 25°C (77°F) was successful in accomplishing this purpose within a variation of ± 0.5°C.

2. The equipment developed for the measurement of volume changes of bituminous-mixture specimens under triaxial compression performed satisfactorily. Some difficulty was experienced in establishing calibration values for the change in volume of the apparatus and the triaxial cell under the application of the various confining pressures.

3. A technique for fabricating four-inch diameter by eight-inch high specimens of a bituminous mixture by the use of the kneading compactor was established. The scheme developed provided specimens that were uniform with respect to density generally within 0.5 percent of the average density of the sections cut from the specimen and uniform with respect to asphalt content to within 0.3 percent of the average asphalt content of the sections.

4. Tests on similar specimens indicated that the three drainage conditions used had no measured effect on the magnitude of observed strength values even though there was indirect evidence of the development of a negative pore pressure during the tests. This finding was established for a five-minute and a one-hour time interval between

the application of axial load increments.

5. It was established that volume changes occur during the testing of the particular bituminous mixtures used and that these changes consist of an initial compression of generally less than one-half of one percent of the specimen volume followed by a continual expansion to failure. This result indicates that dilatancy occurs in dense-graded bituminous mixtures in a fashion similar to that observed in dense-granular soils.

6. For the particular bituminous mixtures tested, a relationship between the void ratio and percent voids of a specimen, at two percent strain and at the Proportional Limit, and the deviator stress at these points was established. This relationship was found to be linear for a semi-log plot, dependent upon the initial voids characteristics of the specimen, and apparently independent of the test confining pressure.

7. The unique character of the voids-deviator stress relationship was verified by tests of specimens compacted to given initial conditions and then having these conditions changed by water saturation prior to testing.

8. The effects of compaction pressure and asphalt content on the apparent shear strength parameters "c" and "Ø", and on the compressive strength, bulk density and aggregate density were investigated for a particular aggregate blend, compaction procedure, and method of test. The findings of this portion of the study show that both the apparent angle of internal friction and cohesion are affected not only by asphalt content but by compaction pressures as well. Indications are

that aggregate density variation provides a fairly good measure of relative values of observed compressive strengths while bulk density does not.

9. Values of the Modulus of Deformation established at the Proportional Limit for the bituminous mixtures tested are presented. The data developed indicate that the Modulus of Deformation was affected only slightly by asphalt content at a low compaction pressure. The effect of asphalt content on specimens prepared at the high compaction pressure was pronounced. The data show that there is a marked effect of compaction pressure on the Modulus of Deformation.

10. The results of a limited investigation of the Deformation Ratio of the bituminous mixtures used indicate that the angle of shearing resistance of a mixture is mobilized gradually with a continuing increase of Deformation Ratio to a point at which the maximum shearing resistance is developed. Further deformation tends to decrease the developed frictional resistance. The maximum angle of shearing resistance defined by the Deformation Ratio studies compares favorably with the apparent angle of shearing resistance established by the Mohr circle of failure analysis.

CONCLUSIONS

The results obtained from the analysis of test data from sixty-five triaxial tests performed on specimens prepared at four different compaction pressures and three different asphalt contents appear to justify the following conclusions. It should be realized that these conclusions are applicable only to the particular bituminous mixtures and compaction procedures, and for triaxial compression tests of the types used in this study. Furthermore, it should be noted that all percent voids relationships are based on a consideration of the asphalt available to fill the mineral aggregate voids rather than upon the total asphalt used in mixing.

Differences in the drainage conditions existing during the performance of triaxial compression tests on dense-graded bituminous mixtures do not materially affect strength values. This conclusion appears justified for time intervals between load applications of one hour or less.

Indirect evidence indicates that a negative pore pressure develops in the gaseous phase of a dense-graded bituminous mixture subjected to triaxial compression. The indicated negative pore pressure did not affect the observed strength results, so it must be concluded that the effect of this pressure is so small as to be masked by the high strength values of the mixtures tested.

All test results clearly demonstrate that the bituminous mixture specimens tested changed volume during the progress of a test and that the volume change consists of a slight reduction in volume followed by an expansion to failure.

Within the range of the materials and procedures used in this study, there appears to be a unique relationship between the percent voids and/or the void ratio and the deviator stress at two percent axial strain and at the Proportional Limit. The established relationship apparently depends upon the initial conditions of void ratio and percent voids to which the specimen is compacted. The results of tests on specimens with the initial conditions varied from the as-compacted values by water saturation appear to justify the established relationship.

Indications of the effect of compaction pressure and asphalt content on the observed apparent shear strength parameters and on the compressive strength of the test mixtures were established. It was found that variation in measured cohesion values depend upon compaction pressure as well as on asphalt content and that the variation with asphalt content becomes more pronounced at the higher compaction pressures. Similar effects are established for the apparent angle of shearing resistance and for compressive strength at failure. It is clearly shown that variations in bulk density do not necessarily reflect similar variations in strength but there are indications that variations in aggregate density are reflected by similar variations in strength.

The Modulus of Deformation defined at the Proportional Limit is affected by asphalt content at high compaction pressures but apparently is independent of asphalt content at low compaction pressures.

A limited number of computations of the Deformation Ratio of the mixtures tested indicates that this measure varies from values below 0.5 to nearly 1.0 during the progress of a test. Further, it is shown that Deformation Ratio values of greater than 0.5 are necessary in order for full frictional shearing resistance to be mobilized.

SUGGESTIONS FOR FURTHER RESEARCH

The triaxial test offers the most versatile tool for the investigation of the fundamental strength characteristics of bituminous mixtures. Numerous variables exist and, at the present time, there is no accepted method of separating these variables and evaluating their effects on strength characteristics. The study herein reported has attempted to open new fields of thought in the analysis of bituminous mixtures. There are numerous investigations that need to be undertaken to further the basic knowledge of such mixtures. The following suggestions constitute only a partial listing of needed research projects.

1. It is apparent to anyone using the kneading compactor that aggregate degradation occurs during the compaction process. The extent of this breakdown, the sizes of particles affected, the part played in the process by the asphalt content, and the relationship of the degradation occurring in laboratory compaction to that during job mixing, placement, and service use should be investigated.

2. Additional testing to verify the unique relationships suggested by this study between void ratio and/or percent voids and deviator stress at a specified stability criterion is needed. In particular, different aggregate blends should be studied to verify whether or not the suggested relationships will hold for mixtures other than the ones used in this investigation. A study similar to the one

reported but utilizing open-graded mixtures would be worthwhile in furthering the knowledge of mixtures of this type.

3. It would be desirable to investigate further the effects of test drainage conditions on strength for increment loading periods in excess of one hour. The apparatus used for the reported study would not be suitable for such long-time test investigations.

4. It would be of particular interest to study the volume change-strength relationships for a bituminous mixture that has yielded a curved envelope of failure under conditions of testing and analysis other than those used in this study such as constant rate of deformation tests. Such an investigation would cast additional light on the explanation of the curved envelope of failure phenomena.

5. Investigations of the relationship of the as-compacted or initial voids characteristics to the value of these characteristics at a suitable strength criterion would be a valuable contribution to the progress of the development of a rational mixture design procedure.

6. The data obtained in this study, as well as that from many others, shows very clearly the need for knowledge of the magnitude of confining pressure that best corresponds to the confinement existing on a section of bituminous mixture subject to full-scale loading. The widespread use of a triaxial test method of mixture design will probably be delayed until such information is available.

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APPENDIX A

TABLE 7

Summary of Results

Tests 1 - 24

Test Number	Type of Test	Confining Pressure, psi	Asphalt Content, %	Compaction Pressure, psi	Initial Conditions			
					Height cm.	Diameter cm.	Density pcf.	Aggregate Density, pcf.
1	Q	15	2.964	150	20.93	10.18	143.27	139.03
2	Q	30	2.964	150	20.90	10.18	143.33	139.09
3	Q	60	2.963	150	20.92	10.18	143.39	139.15
4	Q	15	3.996	150	20.83	10.18	145.27	139.46
5	Q	30	3.999	150	20.70	10.18	146.56	140.52
6	Q	60	3.996	150	20.70	10.18	146.50	140.46
7	Qc	15	3.993	150	20.75	10.18	146.00	139.90
8	Qc	30	3.994	150	20.88	10.18	145.56	139.53
9	Qc	60	3.997	150	20.70	10.18	145.56	140.46
10	CP	15	3.997	150	20.83	10.18	145.50	139.46
11	CP	30	3.997	150	20.73	10.18	146.06	140.03
12	CP	60	3.997	150	20.78	10.18	145.75	139.71
13	Q	15	5.004	150	20.73	10.18	147.14	139.78
14	Q	30	5.001	150	20.64	10.18	148.25	140.59
15	Q	60	5.004	150	20.60	10.18	148.44	140.77
16	Qc	15	5.005	150	20.68	10.18	147.76	140.34
17	Qc	30	5.006	150	20.67	10.18	147.70	140.34
18	Qc	60	5.006	150	20.60	10.18	148.14	140.71
19	CP	15	5.006	150	20.72	10.18	147.26	139.90
20	CP	30	5.006	150	20.72	10.18	147.33	139.90
21	CP	60	5.006	150	20.60	10.18	148.01	140.59
22	Q	15	2.956	250	20.57	10.18	146.14	141.83
23	Q	30	2.961	250	20.53	10.18	146.14	141.83
24	Q	60	2.958	250	20.60	10.17	146.20	141.83

TABLE 7 (cont'd)

Summary of Results

Tests 25 - 48

Test Number	Type of Test	Confining Pressure, psi	Asphalt Content, %	Compaction Pressure, psi	Initial Conditions			
					Height cm.	Diameter cm.	Density pcf.	Aggregate Density, pcf.
25	Q	15	3.998	250	20.41	10.18	148.01	142.08
26	Q	30	3.997	250	20.41	10.18	148.20	142.27
27	Q	60	3.994	250	20.41	10.18	148.26	142.33
28	Q	15	5.009	250	20.19	10.18	151.01	143.39
29	Q	30	5.009	250	20.13	10.18	151.44	143.83
30	Q	60	5.007	250	20.17	10.18	151.07	143.52
31	Q	15	2.968	400	20.32	10.18	147.39	143.02
32	Q	30	2.968	400	20.25	10.18	147.95	143.58
33	Q	60	2.967	400	20.24	10.18	148.07	143.64
34	Q	15	3.997	400	20.22	10.18	149.70	143.71
35	Q	30	4.020	400	20.17	10.18	150.07	144.02
36	Q	60	3.996	400	20.08	10.18	150.51	144.52
37	Qc	15	3.995	400	20.10	10.18	150.63	144.64
38	Qc	30	3.993	400	20.10	10.18	150.45	144.46
39	Qc	60	3.996	400	20.03	10.18	150.95	144.95
40	CP	15	3.996	400	20.10	10.18	150.51	144.52
41	CP	30	3.996	400	20.15	10.18	150.13	144.14
42	CP	60	3.997	400	20.12	10.18	150.38	144.39
43	Q	15	5.005	400	19.90	10.19	152.94	145.27
44	Q	30	5.006	400	19.90	10.19	152.82	145.20
45	Q	60	5.006	400	19.88	10.19	152.82	145.14
46	Qc	15	5.003	400	19.94	10.19	152.69	145.02
47	Qc	30	5.005	400	19.88	10.19	152.94	145.27
48	Qc	60	5.006	400	19.87	10.19	153.00	145.33

TABLE 7 (cont'd)

Summary of Results

Tests 49 - 65

Test Number	Type of Test	Confining Pressure, psi	Asphalt Content, %	Compaction Pressure, psi	Initial Conditions			
					Height cm.	Diameter cm.	Density pcf.	Aggregate Density, pcf.
49	CP	15	5.005	400	19.92	10.19	152.82	145.14
50	CP	30	5.006	400	19.88	10.19	152.88	145.27
51	CP	60	5.006	400	19.92	10.19	152.94	145.27
52	Q	15	2.968	500	20.00	10.18	149.82	145.39
53	Q	30	2.967	500	19.97	10.18	150.01	145.58
54	Q	60	2.967	500	19.96	10.18	150.13	145.64
55	Q	15	3.999	500	19.83	10.18	152.32	146.20
56	Q	30	3.999	500	19.82	10.18	152.51	146.39
57	Q	60	3.999	500	19.83	10.18	152.51	146.45
58	Q	15	5.007	500	19.80	10.18	153.69	146.02
59	Q	30	5.010	500	19.77	10.19	153.69	146.02
60	Q	60	5.010	500	19.77	10.19	153.69	145.95
61	Q*	15	3.996	400	20.15	10.25	150.63	142.08
62	Q*	60	3.999	400	20.18	10.24	150.76	142.15
63	Q**	30	3.995	400	20.02	10.18	151.07	145.02
64	Qc**	30	3.996	400	20.01	10.18	151.26	145.27
65	CP**	30	3.996	400	20.09	10.18	150.57	144.58

* Water saturated prior to testing

** Slow rate of loading

TABLE 7 (cont'd)

Summary of Results

Tests 1 - 24

Test Number	Compressive Strength, psi				Axial Strain, %			
	P.L.	2% str.	failure	0% vol. str.	P.L.	failure	min. vol.	0% vol. str.
1	120	172	182.5	150	0.64	3.77	0.24	0.64
2	145	223	231	239	0.53	3.66	0.31	0.80
3	192.5	320	341	401	0.55	4.00	0.45	1.32
4	112.5	180	190	125	0.58	3.33	0.26	0.68
5	147.5	238	252.5	190	0.60	3.90	0.30	0.92
6	192.5	322	357.5	292.5	0.50	4.08	0.47	1.41
7	115	185	195	140	0.60	4.83	0.25	0.83
8	125	242.5	253	193	0.45	3.92	0.37	0.97
9	200	332.5	360	293	0.56	4.15	0.30	1.24
10	110	180	190	147	0.63	3.45	0.22	1.00
11	145	237.5	247	207	0.60	3.80	0.45	1.17
12	200	322	342.5	310	0.60	3.90	0.40	1.66
13	100	179	186	125	0.54	4.15	0.20	0.74
14	132.5	222.5	233	177.5	0.56	3.44	0.32	0.98
15	197.5	313	340	300	0.65	4.40	0.47	1.67
16	107.5	195	201	158	0.51	2.94	0.20	0.98
17	130.5	237	252	192.5	0.52	3.84	0.35	1.02
18	202.5	331	358	315	0.60	4.69	0.40	1.65
19	115	182.5	188	135	0.60	3.45	0.27	0.81
20	135	227	249	197	0.60	4.45	0.30	1.15
21	210	345	378	351	0.64	4.10	0.45	2.15
22	102.5	202	203	132.5	0.35	2.40	0.20	0.48
23	145	262	266	198	0.42	2.90	0.30	0.68
24	202.5	362	372	313	0.44	3.15	0.41	1.00

TABLE 7 (cont'd)

Summary of Results

Tests 25 - 48

Test Number	Compressive Strength, psi				Axial Strain, %			
	P.L.	2% str.	failure	0% vol. str.	P.L.	failure	min. vol.	0% vol. str.
25	120	206	208	138	0.45	4.50	0.21	0.56
26	150	262	267.5	209	0.49	4.40	0.32	0.86
27	220	373	385	331	0.56	4.30	0.46	1.20
28	115	203	207	142	0.45	3.15	0.25	0.64
29	150	257.5	261	198.5	0.50	2.60	0.30	0.73
30	207.5	365	382	327.5	0.54	3.90	0.44	1.31
31	165	245	247.5	121	0.57	2.20	0.26	0.51
32	195	323	327	177.5	0.58	2.66	0.25	0.67
33	237	441	447	289	0.50	2.64	0.40	1.31
34	165	259	262.5	162.5	0.50	2.90	0.20	0.49
35	205	324	334	237.5	0.52	3.65	0.25	0.66
36	285	442.5	452	395	0.60	3.16	0.40	1.17
37	160	250	251	172.5	0.53	3.44	0.19	0.60
38	202.5	322	325	250	0.52	3.39	0.25	0.76
39	290	440	450	371	0.60	3.12	0.40	1.04
40	170	250	251	185	0.51	2.65	0.17	0.59
41	230	326	331	251	0.62	3.00	0.25	0.72
42	290	439	450	379	0.60	3.40	0.34	1.05
43	98	182	224	114	0.73	4.33	0.31	0.93
44	157.5	250	272	180	0.88	4.15	0.38	1.05
45	205	340	421	331	0.92	4.28	0.53	1.90
46	97.5	190	225	140	0.69	4.45	0.35	1.19
47	155	237.5	278	203	1.05	5.10	0.55	1.52
48	205	323	405	318	1.03	4.73	0.63	1.95

TABLE 7 (cont'd)

Summary of Results

Tests 49 - 65

Test Number	Compressive Strength, psi				Axial Strain, %			
	P.L.	2% str.	failure	0% vol. str.	P.L.	failure	min. vol.	0% vol. str.
49	97.5	198	220	137.5	0.70	4.19	0.34	1.06
50	165	253	276	205	0.93	4.20	0.40	1.25
51	207.5	302.5	392	326	1.17	5.00	0.60	2.30
52	210	298	298	132.5	0.50	2.00	0.15	0.43
53	255	382.5	385	276	0.56	2.60	0.25	0.63
54	360	548	568	452	0.67	2.85	0.37	1.00
55	167	302	305	205	0.41	1.65	0.20	0.56
56	215	371	374	270	0.55	2.44	0.34	0.74
57	315	519	525	421	0.64	2.70	0.48	1.01
58	115	143	196	108	1.49	5.10	0.38	1.40
59	140	172.5	252.5	156	1.56	5.85	0.60	1.77
60	213	215	360	258	1.96	5.85	0.87	2.56
61	130	205	220	180	0.90	3.75	0.50	1.47
62	210	370	404	385	0.76	3.50	0.70	2.30
63	167.5	279	282.5	248	0.45	3.40	0.30	0.88
64	170	275	278	241	0.50	3.15	0.29	1.00
65	160	274	277	246	0.44	3.00	0.35	0.95

TABLE 7 (cont'd)

Summary of Results

Tests 1 - 24

Test Number	Volume Change - cc, (%)			Percent Voids			Void Ratio		
	P.L.	2% str.	min. vol.	initial	P.L.	2% str.	initial	P.L.	2% str.
1	-1.0(+0.01)	+22.0(+1.28)	-2.8(-0.15)	9.97	9.82	11.12	0.1766	0.1745	0.1915
2	-2.0(-0.12)	+14.4(+0.85)	-3.0(-0.18)	9.94	9.60	10.40	0.1761	0.1717	0.1827
3	-4.2(-0.24)	+ 5.6(+0.33)	-4.4(-0.25)	9.91	9.40	9.65	0.1758	0.1675	0.1726
4	-0.7(-0.05)	+20.0(+1.40)	-2.3(-0.14)	7.31	7.08	8.25	0.1731	0.1705	0.1834
5	-2.6(+0.14)	+12.2(+0.81)	-3.3(-0.20)	6.62	6.26	7.16	0.1645	0.1620	0.1708
6	-4.5(-0.28)	+ 5.0(+0.28)	-4.5(-0.28)	6.67	5.79	6.30	0.1650	0.1541	0.1640
7	-2.5(-0.14)	+16.3(+1.03)	-4.5(-0.27)	7.01	6.72	7.62	0.1693	0.1659	0.1760
8	-3.9(-0.26)	+11.0(+0.74)	-4.5(-0.26)	7.27	6.75	7.65	0.1725	0.1664	0.1800
9	-4.2(-0.30)	+ 6.0(+0.41)	-4.9(-0.30)	6.62	5.80	6.40	0.1645	0.1545	0.1630
10	-4.5(-0.31)	+14.8(+0.89)	-6.5(-0.42)	7.29	6.75	7.85	0.1730	0.1662	0.1792
11	-3.5(-0.24)	+12.4(+0.70)	-4.2(-0.25)	6.94	6.47	7.23	0.1685	0.1628	0.1724
12	-6.9(-0.42)	+ 3.2(+0.20)	-7.3(-0.44)	7.15	5.95	6.49	0.1712	0.1559	0.1632
13	-2.2(-0.12)	+17.3(+1.01)	-6.1(-0.22)	4.69	4.05	5.25	0.1704	0.1638	0.1766
14	-3.0(-0.18)	+10.6(+0.67)	-3.9(-0.23)	4.14	3.57	4.45	0.1635	0.1566	0.1669
15	-6.5(-0.38)	+ 2.0(+0.18)	-6.7(-0.41)	4.02	3.01	3.52	0.1621	0.1501	0.1562
16	-5.0(-0.29)	+14.8(+0.94)	-6.5(-0.35)	4.32	3.80	4.90	0.1658	0.1596	0.1740
17	-4.0(-0.24)	+12.5(+0.75)	-4.5(-0.26)	4.32	3.67	4.57	0.1658	0.1581	0.1712
18	-6.5(-0.38)	+ 3.0(+0.20)	-6.8(-0.41)	4.04	3.00	3.58	0.1625	0.1501	0.1569
19	-2.1(-0.13)	+16.0(+1.08)	-3.9(-0.23)	4.63	4.24	5.15	0.1697	0.1642	0.1790
20	-4.0(-0.26)	+10.0(+0.59)	-5.5(-0.32)	4.60	3.97	4.64	0.1692	0.1616	0.1740
21	-9.4(-0.56)	- 0.05(-0.07)	-9.7(-0.58)	4.15	2.97	3.47	0.1638	0.1492	0.1561
22	-1.5(-0.09)	+28.0(+1.55)	-2.2(-0.14)	8.20	7.94	9.52	0.1536	0.1504	0.1702
23	-2.5(-0.15)	+19.1(+1.14)	-2.6(-0.16)	8.19	7.85	9.10	0.1537	0.1495	0.1642
24	-3.5(-0.21)	+10.1(+0.60)	-3.5(-0.21)	8.17	7.84	8.33	0.1534	0.1493	0.1586

TABLE 7 (cont'd)

Summary of Results

Tests 25 - 48

Test Number	Volume Change - cc. (%)			Percent Voids			Void Ratio		
	P.L.	2% str.	min. vol.	initial	P.L.	2% str.	initial	P.L.	2% str.
25	-0.9(-0.06)	+24.8(+1.52)	-2.1(-0.13)	5.54	5.34	6.80	0.1512	0.1490	0.1665
26	-3.1(-0.18)	+16.6(+1.01)	-3.5(-0.21)	5.45	5.08	6.20	0.1500	0.1458	0.1660
27	-4.1(-0.25)	+ 8.5(+0.51)	-4.3(-0.26)	5.40	4.94	5.66	0.1494	0.1438	0.1525
28	-1.0(-0.07)	+20.0(+1.26)	-1.9(-0.12)	2.22	1.86	3.23	0.1408	0.1368	0.1512
29	-1.8(-0.10)	+15.1(+0.92)	-2.5(-0.15)	1.93	1.76	2.75	0.1374	0.1356	0.1478
30	-5.0(-0.30)	+ 5.9(+0.36)	-5.2(-0.31)	2.14	1.61	2.26	0.1399	0.1339	0.1415
31	+0.6(+0.04)	+27.5(+1.69)	-1.9(-0.12)	7.38	7.35	9.52	0.1438	0.1432	0.1540
32	-1.0(-0.07)	+23.6(+1.41)	-2.6(-0.16)	7.04	6.82	9.30	0.1396	0.1369	0.1493
33	-6.4(-0.39)	+ 9.0(+0.55)	-6.5(-0.40)	6.99	6.45	7.30	0.1389	0.1324	0.1439
34	0.0(0.00)	+29.5(+1.82)	-2.1(-0.12)	4.50	4.49	6.10	0.1386	0.1388	0.1586
35	-1.4(-0.08)	+21.7(+1.32)	-2.8(-0.16)	4.22	3.85	5.80	0.1359	0.1316	0.1540
36	-4.8(-0.28)	+10.8(+0.64)	-5.3(-0.34)	3.95	3.42	4.30	0.1321	0.1257	0.1362
37	-0.3(-0.05)	+28.2(+1.73)	-3.1(-0.18)	3.87	3.70	5.10	0.1311	0.1290	0.1442
38	-2.5(-0.15)	+21.7(+1.33)	-4.0(-0.23)	3.99	3.59	5.00	0.1325	0.1277	0.1436
39	-4.4(-0.26)	+12.3(+0.76)	-5.3(-0.32)	3.67	3.34	4.30	0.1288	0.1249	0.1358
40	-0.5(-0.05)	+28.7(+1.74)	-3.1(-0.19)	3.97	3.90	5.10	0.1323	0.1314	0.1450
41	-1.5(-0.07)	+22.1(+1.33)	-3.7(-0.22)	4.22	3.95	5.30	0.1352	0.1321	0.1476
42	-4.5(-0.26)	+12.1(+0.61)	-5.4(-0.32)	4.04	3.58	4.52	0.1331	0.1276	0.1386
43	-1.2(-0.07)	+10.3(+0.57)	-2.3(-0.27)	0.96	0.73	1.44	0.1263	0.1238	0.1315
44	-1.0(-0.07)	+ 9.6(+0.55)	-2.3(-0.27)	1.01	0.75	1.40	0.1269	0.1239	0.1311
45	-5.1(-0.24)	+ 0.6(+0.05)	-5.5(-0.34)	1.02	0.60	0.95	0.1270	0.1222	0.1262
46	-2.6(-0.18)	+10.2(+0.62)	-3.6(-0.23)	1.11	0.90	1.66	0.1280	0.1259	0.1346
47	-3.4(-0.19)	+ 4.6(+0.28)	-4.7(-0.28)	0.96	0.60	1.07	0.1263	0.1220	0.1274
48	-4.9(-0.29)	+ 0.4(+0.03)	-5.1(-0.32)	0.90	0.54	0.85	0.1256	0.1216	0.1249

TABLE 7 (cont'd)

Summary of Results

Tests 49 - 65

Test Number	Volume Change - cc. (%)			Percent Voids			Void Ratio		
	P.L.	2% str.	min. vol.	initial	P.L.	2% str.	initial	P.L.	2% str.
49	-2.5(-0.16)	+11.0(+0.61)	-3.3(-0.20)	1.04	0.88	1.67	0.1271	0.1251	0.1344
50	-2.2(-0.14)	+ 8.0(+0.49)	-3.8(-0.24)	0.97	0.78	1.40	0.1264	0.1242	0.1314
51	-5.9(-0.35)	- 1.8(-0.06)	-6.6(-0.40)	0.96	0.45	0.67	0.1263	0.1204	0.1230
52	+0.7(+0.05)	+40.6(+2.50)	-1.8(-0.10)	5.87	5.87	8.10	0.1254	0.1254	0.1528
53	-0.5(-0.04)	+29.5(+1.78)	-2.4(-0.15)	5.74	5.66	7.36	0.1238	0.1228	0.1435
54	-3.2(-0.20)	+16.9(+1.02)	-4.1(-0.25)	5.69	5.44	6.59	0.1233	0.1203	0.1341
55	-1.1(-0.08)	+33.5(+2.05)	-1.7(-0.12)	2.81	2.71	4.76	0.1191	0.1178	0.1418
56	-1.4(-0.10)	+23.7(+1.50)	-2.0(-0.15)	2.68	2.51	4.03	0.1174	0.1158	0.1340
57	-2.9(-0.19)	+14.6(+0.93)	-3.3(-0.20)	2.68	2.40	3.48	0.1174	0.1144	0.1264
58	+0.6(+0.03)	+ 3.8(+0.24)	-2.1(-0.13)	0.45	0.42	0.63	0.1206	0.1203	0.1226
59	-1.1(-0.06)	+ 0.9(+0.08)	-2.8(-0.16)	0.45	0.36	0.48	0.1207	0.1197	0.1210
60	-2.5(-0.15)	- 2.4(-0.14)	-4.4(-0.26)	0.47	0.27	0.28	0.1208	0.1186	0.1186
61	-6.0(-0.40)	+ 8.0(+2.26)	-7.7(-0.47)	1.28	0.84	1.70	0.1517	0.1462	0.1564
62	-11.8(-0.71)	- 3.3(-0.20)	-12.0(-0.72)	1.20	0.37	0.90	0.1512	0.1415	0.1474
63	-4.5(-0.30)	+19.0(+1.15)	-4.8(-0.30)	3.61	3.25	4.65	0.1280	0.1240	0.1403
64	-5.4(-0.35)	+16.4(+0.99)	-6.0(-0.36)	3.47	3.07	3.98	0.1317	0.1219	0.1368
65	-5.3(-0.32)	+19.0(+1.15)	-5.5(-0.34)	3.92	3.49	4.91	0.1265	0.1266	0.1435

TABLE 8

Modulus of Deformation Data

(as plotted in Figure 15)

Type of Test	Confining Pressure psi	Asphalt Content %	Compaction Pressure psi	Proportional Limit,psi	Strain at P.L.,%	C_d,psi
Q	15	4	150	112.5	0.58	19400
Q	30	4	150	147.5	0.60	25580
Q	60	4	150	192.5	0.50	38500
Qc	15	4	150	115	0.60	19170
Qc	30	4	150	125	0.45	27780
Qc	60	4	150	200	0.56	35710
CP	15	4	150	110	0.63	17460
CP	30	4	150	145	0.60	24170
CP	60	4	150	200	0.60	33330
Q	15	5	150	100	0.54	18520
Q	30	5	150	132.5	0.56	23660
Q	60	5	150	197.5	0.65	30380
Qc	15	5	150	107.5	0.51	21080
Qc	30	5	150	130.5	0.52	25100
Qc	60	5	150	202.5	0.60	33750
CP	15	5	150	115	0.60	19170
CP	30	5	150	135	0.60	22500
CP	60	5	150	210	0.64	32810
Q	15	4	400	165	0.50	33000
Q	30	4	400	205	0.52	39420
Q	60	4	400	285	0.60	47500
Qc	15	4	400	160	0.53	30190
Qc	30	4	400	202.5	0.52	38940
Qc	60	4	400	290	0.60	48330
CP	15	4	400	170	0.51	33330
CP	30	4	400	230	0.62	37100
CP	60	4	400	290	0.60	48330
Q	15	5	400	98	0.73	13420
Q	30	5	400	157.5	0.88	17900
Q	60	5	400	205	0.92	22280
Qc	15	5	400	97.5	0.69	14130
Qc	30	5	400	155	1.05	14760
Qc	60	5	400	205	1.03	19900
CP	15	5	400	97.5	0.70	13930
CP	30	5	400	165	0.93	17740
CP	60	5	400	207.5	1.17	17740

TABLE 9

Modulus of Deformation Data

Type of Test	Confining Pressure psi	Asphalt Content %	Compaction Pressure psi	Proportional Limit,psi	Strain at P.L.,%	C_d,psi
Q	15	3	150	120	0.64	18750
Q	30	3	150	145	0.53	27360
Q	60	3	150	192.5	0.55	35000
Q	15	3	250	102.5	0.35	29290
Q	30	3	250	145	0.42	34520
Q	60	3	250	202.5	0.44	46020
Q	15	4	250	120	0.45	26670
Q	30	4	250	150	0.49	30610
Q	60	4	250	220	0.56	39280
Q	15	5	250	115	0.45	25550
Q	30	5	250	150	0.50	30000
Q	60	5	250	207.5	0.54	38420
Q	15	3	400	165	0.57	28950
Q	30	3	400	195	0.58	33620
Q	60	3	400	237	0.50	47400
Q	15	3	500	210	0.50	42000
Q	30	3	500	255	0.56	45530
Q	60	3	500	360	0.67	53730
Q	15	4	500	167	0.41	40730
Q	30	4	500	215	0.55	39090
Q	60	4	500	315	0.64	49220
Q	15	5	500	115	1.49	7720
Q	30	5	500	140	1.56	8970
Q	60	5	500	213	1.96	10870
Q*	15	4	400	130	0.90	14440
Q*	60	4	400	210	0.76	27630
Q**	30	4	400	167.5	0.45	37200
Qc**	30	4	400	170	0.50	34000
CP**	30	4	400	160	0.44	36390

* Water saturated prior to testing
** Slow rate of loading

TABLE 10

Shear Strength Parameters at Failure

Asphalt Content, %	Compaction Pressure, psi	Cohesion "c", psi	Angle of Internal Friction, "Ø" degrees
3	150	32	39.3
3	250	34	41.0
3	400	40	43.0
3	500	40	48.5
4	150	32*	40.3*
4	250	34	41.5
4	400	43*	42.4*
4	500	48	45.2
5	150	32*	40.8*
5	250	34	41.2
5	400	37.5*	41.8*
5	500	33	40

* Average values from the results of "Q", "Qc", and "CP" tests at the indicated asphalt contents and compaction pressures

TABLE 11

Deformation Ratio Results

	4% Asphalt Content			
	Compaction Pressure 150 psi		Compaction Pressure* 400 psi	
Confining Pressure psi	D_r	Sine ϕ_1	D_r	Sine ϕ_1
15	0.63	0.121	0.75	0.584
	0.72	0.424	0.94	0.682
	0.81	0.563	1.03	0.654
	0.90	0.621		
	0.94	0.635		
30	0.48	0.077	0.57	0.428
	0.56	0.414	0.64	0.574
	0.64	0.565	0.75	0.658
	0.71	0.612	0.92	0.705
	0.78	0.641	0.98	0.688
	0.83	0.642		
60	0.37	0.048	0.40	0.277
	0.42	0.307	0.43	0.409
	0.47	0.454	0.47	0.500
	0.51	0.532	0.52	0.566
	0.58	0.588	0.57	0.617
	0.62	0.609	0.64	0.654
	0.65	0.627	0.72	0.676
	0.69	0.640	0.81	0.681
Range of Maximum ϕ_1, degrees		39.4-40.2		43.0-44.8

* Data plotted in Figure 34

TABLE 12

Deformation Ratio Results

Confining Pressure psi	5% Asphalt Content			
	Compaction Pressure 150 psi		Compaction Pressure 400 psi	
	D_r	Sine ϕ_1	D_r	Sine ϕ_1
15	0.63	0.249	0.64	0.335
	0.77	0.598	0.69	0.514
	0.91	0.628	0.75	0.612
			0.82	0.634
			0.87	0.644
30	0.50	0.152	0.55	0.296
	0.61	0.486	0.62	0.571
	0.71	0.601	0.65	0.617
	0.79	0.620	0.71	0.652
	0.80	0.638	0.77	0.677
			0.83	0.677
60	0.36	0.089	0.45	0.232
	0.38	0.333	0.47	0.407
	0.44	0.468	0.50	0.477
	0.51	0.541	0.53	0.563
	0.58	0.595	0.55	0.589
	0.63	0.615	0.57	0.612
	0.67	0.629	0.59	0.632
	0.71	0.630	0.62	0.650
			0.64	0.665
			0.68	0.677
			0.72	0.679
Range of Maximum ϕ_1, degrees		38.7-39.5		40.2-42.9

APPENDIX B

TABLE 13

Typical Mix Record

Specimen Q-12

	Batch 1	Batch 2
Tare Weight of Bowl, g	3141	3140
Weight of Bowl + Aggregate, g	5039	5038
Weight of Aggregate, g	1898	1898
Weight of Bowl + Aggregate + Asphalt, g	5139	5138
Weight of Asphalt, g	100	100
Weight of Mixture, g	1998	1998
Asphalt Content of Batch, %		
by weight of mixture	5.005	5.005
by weight of aggregate	5.269	5.269

Asphalt Content of Specimen, %	
by weight of mixture	5.005
by weight of aggregate	5.269

Specimen Diameter, cm	10.19, 10.19, 10.19 : Average = 10.19
Specimen Height, cm	19.95, 19.90, 19.85 : Average = 19.90
Specimen Volume, cc	1622.85
Specimen Weight, g	3997.0
Specimen Density, g/cc	2.451
Specimen Density, pcf	152.942

Date: 3/21/60
By: JS & TC

TABLE 14

Typical Specimen Initial Characteristics Computations

Specimen Q-12

Asphalt Content, % by weight of mix	5.005
Volume of Specimen, cc	1622.85
Weight of Specimen, g	3977.0
Weight of Aggregate, g	3777.951
Weight of Asphalt, g	199.049
% Bitumen Absorption	0.72
Weight of Asphalt Absorbed, g	27.201
Net Weight of Asphalt, g	171.848
Specific Gravity, Aggregate Blend	2.622
Specific Gravity, Asphalt	1.033
Volume of Aggregate (V_m), cc	1440.87
Volume of Asphalt (V_b), cc	166.36
Volume of Aggregate + Asphalt, cc	1607.23
Volume of Air (V_a), cc	15.62
Volume of Voids in Aggregate, cc	181.98
% V	0.96
$\%V_{ma}$	11.21
% V filled	91.42
$e = \frac{V_a + V_b}{V_m}$	0.1263
Length of Specimen, cm	19.90
Cross-Sectional Area, sq. cm	81.55
Aggregate Density, g/cc	2.328
Aggregate Density, pcf	145.267

Date: 3/22/60
By: JS

TABLE 15

Typical Test Data

Type of Test: "Q"
Specimen: Q-12, 5%, 400 psi

Date: 3/23/60
By: JS

Loading psi *	Elapsed Time, min.	Vol. Scale cm	δ Vol. Scale, cm	δ Vol. cc	Corr. δV, cc	ΔV cc	Dial cm	ΔDial cm	Piston cc
							0.043		
σ_3 = 15	0	37.50							
	0.25	33.40	-4.10	-20.377	-2.465	-2.465	0.035	0.008	-0.023
	5.0	33.05	-0.35	- 1.740	-1.740	-4.205	0.035		
Δσ = 50	10.0	33.00	-0.05	- 0.248	-0.583	-4.788	0.153	0.118	+0.335
100	15.0	33.30	+0.30	+ 1.491	+1.272	-3.516	0.236	0.077	-0.219
150	20.0	34.00	+0.70	+ 3.479	+3.203	-0.313	0.333	0.097	-0.276
180	25.0	34.95	+0.95	+ 4.722	+4.469	+4.156	0.422	0.089	-0.253
200	30.0	36.20	+1.25	+ 6.212	+5.945	+10.101	0.516	0.094	-0.267
220	35.0	38.35	+2.15	+10.686	+10.302	+20.403	0.651	0.135	-0.384
230	40.0	41.80	+3.45	+17.146	+16.580	+36.983	0.850	0.199	-0.566
240	43.0	47.00	+5.20	+25.844	+24.991	+61.974	1.150	0.300	-0.853

* Nominal stress on a 4-inch diameter specimen

TABLE 16

Typical Stress and Strain Computations

Type of Test: "Q" Date: 3/23/60
Specimen: Q-12, 5%, 400 psi By: JS

Total Load lbs	Friction lbs	Net Load lbs	ΔVol. cc	$V_1 + \Delta V$ cc	Δ1 cm	$l_1 - \Delta l$ cm	Area sq. cm	Stress psi	Strain %
σ_3 = 15 psi				1622.85		19.90	81.55		
at 15 sec			- 2.465	1620.38					
at 5 min			- 4.205	1618.64					
628	20	608	- 4.788	1618.06	0.110	19.79	81.76	48.0	0.553
1257	30	1227	- 3.516	1619.33	0.077	19.71	82.16	96.3	0.940
1885	46	1839	- 0.313	1622.54	0.097	19.62	82.70	143.4	1.427
2263	59	2204	+ 4.156	1627.01	0.089	19.53	83.31	170.6	1.874
2514	74	2440	+10.101	1632.95	0.094	19.43	84.04	187.2	2.347
2765	87	2678	+20.403	1643.25	0.135	19.30	85.14	202.9	3.025
2891	95	2796	+36.983	1659.83	0.199	19.10	86.90	207.5	4.205
3016	101	2915	+61.974	1684.82	0.300	18.80	89.62	209.8	5.533

TABLE 17

Typical Volume Change, Void Ratio and Percent Voids Computations

Type of Test: "Q"
Specimen: Q-12, 5%, 400 psi

Date: 3/23/60
By: JS

Volume of Asphalt = 166.36 cc
Volume of Aggregate = 1440.87 cc

Stress psi	ΔVol. cc	ΔVol. - V_{sv}* cc	Specimen Vol.,cc	Vol. Air cc	% Change in Vol.	Voids %	Void Ratio
			1622.85	15.62		0.96	0.1263
0		- 2.465	1620.38	13.15		0.81	0.1246
15.0	- 4.205	- 1.740	1618.64	11.42	-0.107	0.71	0.1234
63.0	- 4.788	- 2.323	1618.06	10.83	-0.143	0.67	0.1230
111.3	- 3.516	- 1.051	1619.33	12.10	-0.065	0.75	0.1238
158.4	- 0.313	+ 2.152	1622.54	15.31	+0.133	0.94	0.1261
185.6	+ 4.516	+ 6.621	1627.01	19.78	+0.409	1.22	0.1292
202.4	+10.101	+12.566	1632.95	25.72	+0.775	1.57	0.1333
217.9	+20.403	+22.868	1643.25	36.02	+1.411	2.19	0.1405
222.5	+36.983	+39.448	1659.83	52.60	+2.434	3.17	0.1520
224.8	+61.974	+64.439	1684.82	77.59	+3.977	4.60	0.1693

* Assumes the specimen volume change represented by the 15 sec. reading is due to surface voids only

TABLE 18

Typical Computations of D_r and Sine ϕ_1

The following computations are based on data from a "Q" test at a confining pressure of 30 psi on a specimen prepared at 4 percent asphalt content and with a 400 psi foot pressure compaction. The volume of the specimen after the application of the confining pressure was 1635.18 cc.

D_r = Deformation Ratio = ratio of lateral strain to axial strain.

Computations of D_r

$\sigma_1 - \sigma_3$ psi	ΔV cc	ϵ_v %	ϵ_a %	$\epsilon_v - \epsilon_a$ %	$2\epsilon_a$ %	D_r
48.31	- 0.888	-0.054	-0.228	+ 0.174	- 0.556	0.31
97.33	- 1.207	-0.074	-0.352	+ 0.278	- 0.704	0.39
145.70	- 1.027	-0.063	-0.471	+ 0.408	- 0.942	0.43
192.90	- 0.107	-0.006	-0.600	+ 0.594	- 1.200	0.49
239.36	+ 1.779	+0.109	-0.778	+ 0.887	- 1.556	0.57
275.41	+ 4.865	+0.297	-1.031	+ 1.328	- 2.062	0.64
309.86	+12.309	+0.753	-1.482	+ 2.235	- 2.964	0.75
338.08	+46.496	+2.843	-3.401	+ 6.244	- 6.802	0.92
327.27	+86.084	+5.264	-5.434	+10.698	-10.864	0.98

ϕ_1 = angle of shearing resistance

Computations of Sine ϕ_1

σ_1,psi	Z*,psi	σ_1-Z,psi	F	(F-1)	(F+1)	Sine ϕ_1
269.36	194.5	74.86	2.495	1.495	3.495	0.428
305.41		110.91	3.697	2.697	4.697	0.574
339.86		145.36	4.845	3.845	5.845	0.658
368.08		173.58	5.786	4.786	6.786	0.705
357.07		162.57	5.419	4.419	6.419	0.688

* $Z = 2c \tan (45 + \phi/2)$

APPENDIX C

TABLE 19

Typical Density Test Results

Foot Pressure, 150 psi

Design Asphalt Content, % by Weight of Mixture	Density, pcf					
	2.8		4.0		5.0	
	Measured	Displaced	Measured	Displaced	Measured	Displaced
Entire Specimen	143.146	146.453	146.640	149.136	149.698	151.320
Top	142.147	145.642	145.018	148.574	148.824	151.320
Middle	142.958	144.830	145.954	148.886	149.011	151.195
Bottom	142.022	146.952	144.893	149.356	149.356	151.819
Maximum % Variation from Average	+0.4,-0.2	+0.8,-0.7	+0.5,-0.3	+0.3,-0.3	+0.3,-0.2	+0.3,-0.2
Foot Pressure, 250 psi						
Entire Specimen	145.018	147.576	147.451	150.509	150.821	152.880
Top	144.394	146.515	147.389	150.010		152.914
Middle	145.267	147.451	148.262	150.134		152.568
Bottom	144.643	149.198	147.014	150.758		153.317
Maximum % Variation from Average	+0.3,-0.3	+1.0,-0.8	+0.4,-0.4	+0.3,-0.2		+0.4,-0.3
Foot Pressure, 400 psi						
Entire Specimen	147.326	149.448	149.885	152.568	153.067	154.066
Top	145.766	148.637	148.949	150.883		153.629
Middle	147.264	149.573	150.883	152.693		154.752
Bottom	145.576	151.133	150.946	153.691		154.315
Maximum % Variation from Average	+0.5,-0.7	+0.9,-0.8	+0.5,-0.9	+0.8,-1.0		+0.3,-0.4

TABLE 20

Typical Asphalt Extraction Test Results

Design Asphalt Content % by Weight of Mixture	Asphalt Content by Extraction % by Weight of Mixture		
	2.8	4.0	5.0
Foot Pressure, 150 psi			
Top	2.9	4.0	4.8
Middle	2.7	3.9	4.9
Bottom	2.7	4.1	5.0
Foot Pressure, 250 psi			
Top	2.7	4.0	4.4
Middle	2.7	3.8	4.5
Bottom	2.9	3.8	4.7
Foot Pressure, 400 psi			
Top	2.8	3.9	4.7
Middle	2.6	3.9	5.2
Bottom	2.5	3.8	4.9

APPENDIX D

TABLE 21

Typical Temperature Cycles

Controlled Temperature Room

Date: 2/14/60

Time	Temperature Reading, °C			Time	Temperature Reading, °C		
	A(1)	B(2)	C(3)		A(1)	B(2)	C(3)
1:10	25.1	25.3	25.1	1:25	24.5	24.8	25.0
	24.2	24.5	24.5		25.2	25.4	25.4
	24.1	24.5	24.4		25.7	26.0	25.9
	24.7	25.0	25.0		26.0	26.2	26.0
	25.3	25.5	25.4		25.9	25.2	25.3
1:15	25.8	26.0	25.8	1:30	24.2	24.5	24.9
	25.8	25.9	25.8		24.4	24.8	25.0
	24.6	24.8	25.0		25.1	25.4	25.4
	24.0	24.4	24.6		25.7	26.0	25.8
	24.5	24.8	24.9		25.9	26.1	26.0
1:20	25.1	25.4	25.3	1:35	24.9	25.1	25.2
	25.7	26.0	25.8		24.2	24.5	24.6
	25.9	26.1	26.0		24.5	24.8	24.9
	24.9	25.1	25.2		25.1	25.4	25.3
	24.1	24.5	24.7		25.6	26.0	25.8
				1:40	25.9	26.2	26.0

Average Temperature, °C at Location A = 25.05
at Location B = 25.30
at Location C = 25.30

Note: Temperature readings of the water in the water reservoir tank did not vary 0.1°C from 25.0°C during the period of observation.

(1) located at the sensing element of the temperature control
(2) located at the position of the triaxial cell
(3) located at the volume measuring device

TABLE 22

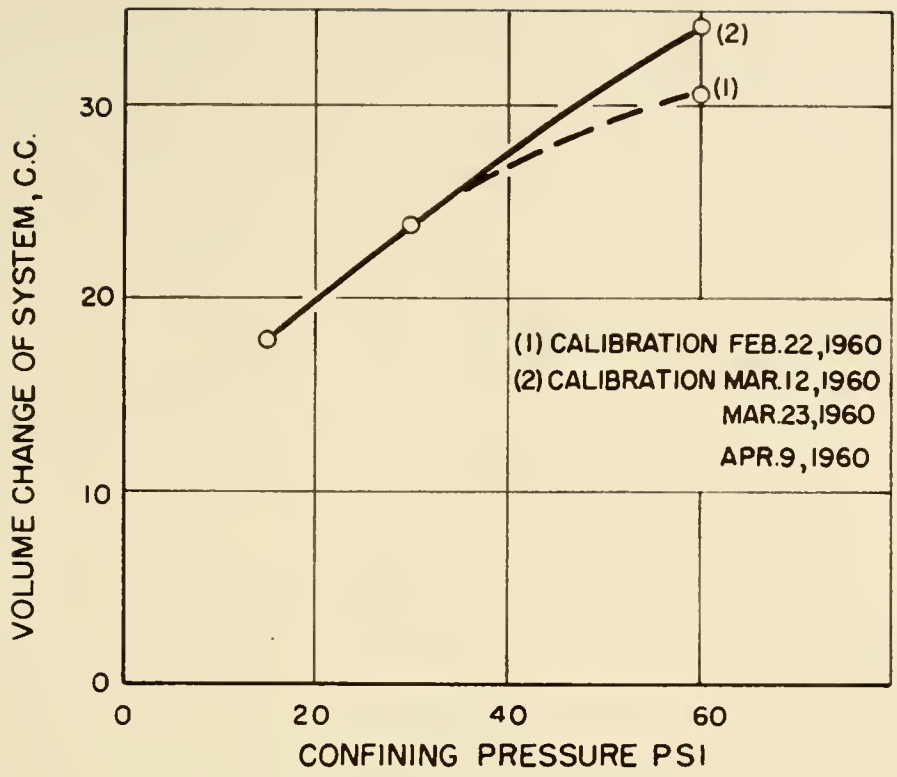
Volume Change Due to Piston

In the performance of a test, a 3/4-inch diameter hardened steel piston is used to apply load to the specimen. As this piston moves into the cell upon deformation of the specimen, the volume of the cell liquid is changed. To compensate for this volume change, a correction must be applied to the observed volume change readings.

The piston correction is computed on the basis of the volume of liquid displaced by the 3/4-inch (1.9025 cm) diameter piston moving a distance of 0.001 cm. Tables were prepared that permitted direct evaluation of the piston displacement resulting from any given extensometer change. The portion of the table for movements of 0.000 to 0.099 cm was used most frequently and is the portion selected as illustrative of this computation and the method of establishing the piston displacement correction.

Volume Change, cc

Deformation / cm	0.000	0.001	0.002	0.003	0.004	0.005	0.006	0.007	0.008	0.009
0.00	-	0.003	0.006	0.008	0.011	0.014	0.017	0.020	0.023	0.025
0.01	0.028	0.031	0.034	0.037	0.040	0.043	0.045	0.048	0.051	0.054
0.02	0.057	0.060	0.062	0.065	0.068	0.071	0.074	0.077	0.080	0.082
0.03	0.085	0.088	0.091	0.094	0.097	0.099	0.102	0.105	0.108	0.111
0.04	0.114	0.116	0.119	0.122	0.125	0.128	0.131	0.134	0.136	0.139
0.05	0.142	0.145	0.148	0.151	0.153	0.156	0.159	0.162	0.165	0.168
0.06	0.170	0.173	0.176	0.179	0.182	0.185	0.188	0.190	0.193	0.196
0.07	0.199	0.202	0.205	0.207	0.210	0.213	0.216	0.219	0.222	0.225
0.08	0.227	0.230	0.233	0.236	0.239	0.242	0.244	0.247	0.250	0.253
0.09	0.256	0.259	0.262	0.264	0.267	0.270	0.273	0.276	0.279	0.281



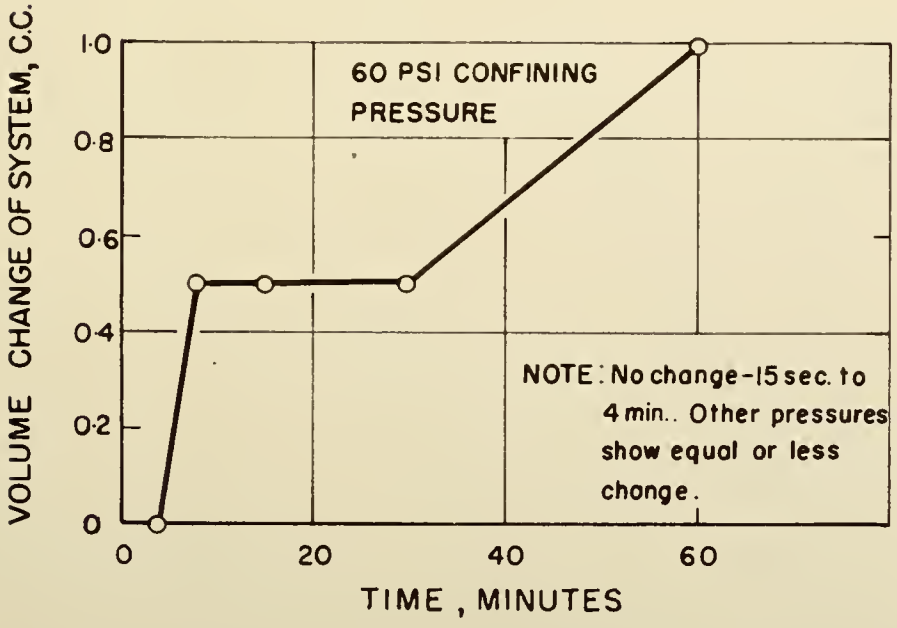


FIGURE 35. CALIBRATION CURVES FOR VOLUME MEASURING DEVICE AND TRIAXIAL CELL

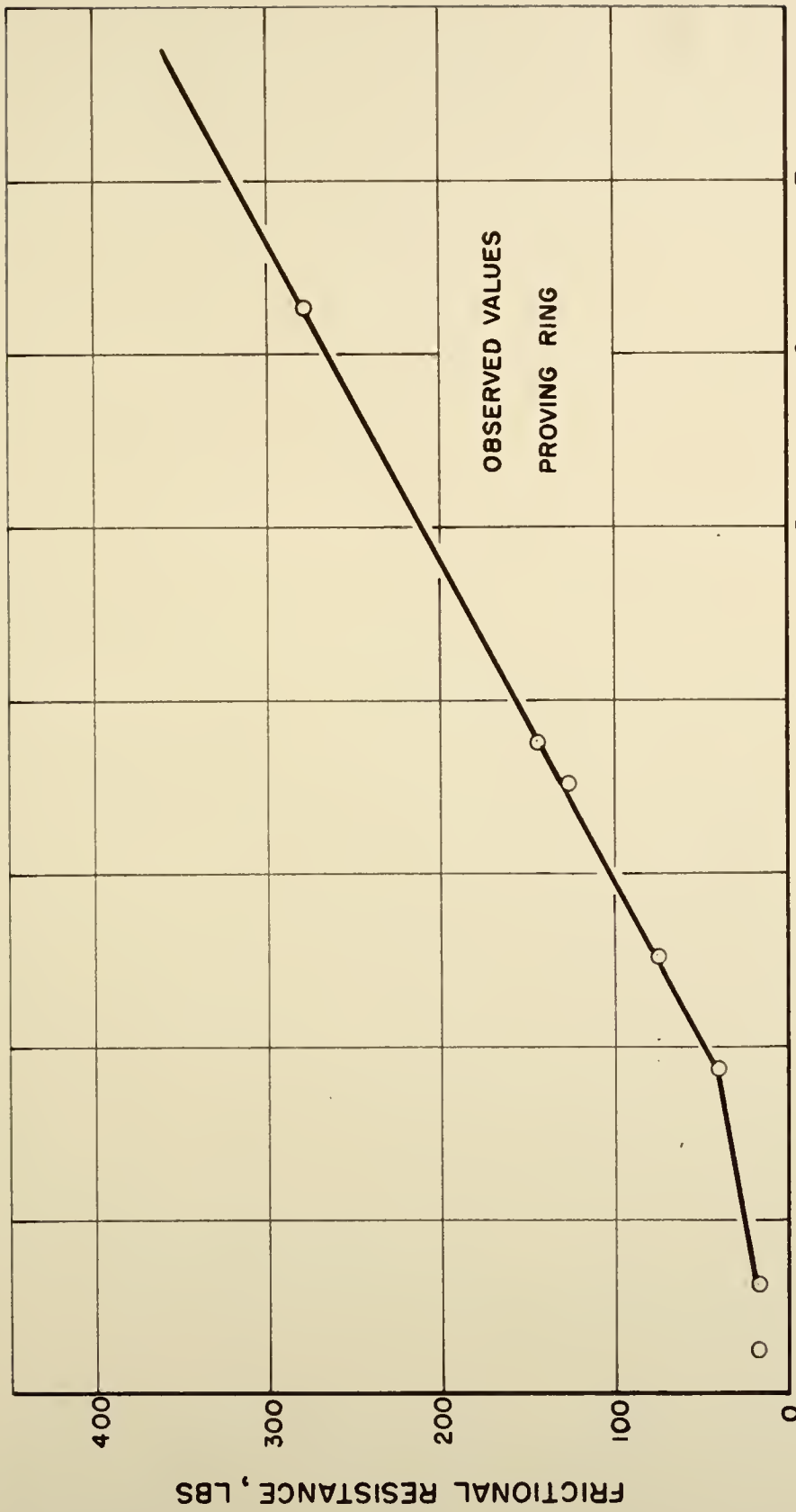
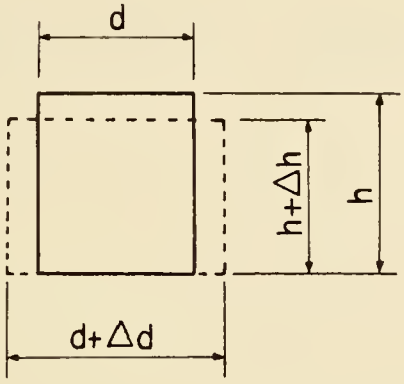


FIGURE 36. FRICTIONAL RESISTANCE CALIBRATION CURVE

APPENDIX E

Deformation Ratio and Angle of Shearing Resistance

1. Consider a right cylinder of initial height h and diameter d and having these dimensions changed by an incremental amount under load.



2. The initial volume of the cylinder = $V = \frac{\pi d^2}{4}(h)$

3. Volume after compression is $V + \Delta V = \frac{\pi}{4}(d + \Delta d)^2 (h + \Delta h)$

4. For small values of Δd and Δh

$$V + \Delta V = V\left(1 + \frac{2\Delta d}{d} + \frac{\Delta h}{h}\right)$$

5. If $\frac{\Delta d}{d}$ is defined as lateral strain (ϵ_1) and $\frac{\Delta h}{h}$ as axial strain (ϵ_a) then $\frac{\Delta V}{V} = 2\ \epsilon_1 + \epsilon_a$

6. Volumetric strain is defined as $\epsilon_v = \frac{\Delta V}{V} = 2\ \epsilon_1 + \epsilon_a$

7. The Deformation Ratio (D_r) is analogous to Poisson's Ratio and is defined in a similar manner as the ratio of lateral strain to axial strain. Therefore $D_r = -\ \epsilon_1 / \epsilon_a$

8. Substituting from Step 6 and using the definition of Step 7

$$D_r = - \frac{\epsilon_v - \epsilon_a}{2 \epsilon_a}$$

9. Friction component of stress

$$\tan^2(45 + \phi/2) = \frac{\sigma_1 - 2c \tan (45 + \phi/2)}{\sigma_3} = (1 + \sin \phi)/(1 - \sin \phi)$$

10. Defining $\frac{(1 + \sin \phi)}{(1 - \sin \phi)} = F$ then $\sin \phi = \frac{(F - 1)}{(F + 1)}$

Relationship Between Percent Voids and Void Ratio

Assumed to be known:

Percent Voids = $v = (V_a/V) \times 100 = \frac{\text{Volume of Air Voids}}{\text{Volume of Specimen}} \times 100$

Asphalt Content, % by weight of mixture = C

$= \frac{\text{total weight of asphalt}}{\text{weight of specimen}} \times 100$

Bulk Density of Specimen = $\gamma = \frac{\text{weight of specimen}}{\text{volume of specimen}}$

Percent Asphalt Absorbed = A

$= \frac{\text{weight of asphalt absorbed by aggregate}}{\text{weight of aggregate}} \times 100$

Specific Gravity of Asphalt = G

Unit Weight of Water = γ_w

Find: Void Ratio in Terms of Known Quantities

By Definition: Void Ratio = $e = \frac{\text{Vol.Air + Vol.Asphalt Filling Voids}}{\text{Vol. Aggregate}}$.

1. considering a unit volume of specimen

   volume of air = $V_a = v/100$

2. total weight of asphalt = $(C/100)\,\gamma$

3. weight of aggregate in specimen = $\gamma - (C/100)\,\gamma$

4. weight of asphalt absorbed = $(\gamma - C\gamma/100)(A/100)$

5. weight of asphalt effective in filling voids

   $= (C\gamma/100)(1 + A/100) - (A\gamma/100)$

6. volume of asphalt filling voids = V_b

   $= \frac{(C\gamma/100)(1 + A/100) - (A\gamma/100)}{G\,\gamma_w}$

7. volume of aggregate = $1 - V_a - V_b$

8. void ratio = e = $(V_a + V_b)/(1 - V_a - V_b)$

   a. note that V_a and V_b are established from steps 1 and 6 respectively.

VITA

VITA

James Hamilton Schaub was born on January 27, 1925, in Moundsville, West Virginia. He attended the grammar and high schools of that city and completed his high school education at the Capitol Page School, Washington, D. C., in June, 1942.

Mr. Schaub attended George Washington University from June, 1942, until March, 1943, as a part-time student while employed in the Post Office of the Senate Office Building.

Mr. Schaub entered the United States Army in April, 1943. Following basic training, he attended Virginia Polytechnic Institute under the ASTP program. In April, 1944, he was assigned to the 334th Infantry Regiment and completed his World War II military service, including combat and occupational duties in Europe, with that unit.

Upon completion of military service, Mr. Schaub re-entered Virginia Polytechnic Institute and received the Bachelor of Science in Civil Engineering degree in June, 1948. He attended Harvard University the following year and was awarded the Master of Science degree in June, 1949.

Mr. Schaub's experience includes three years employment as Chief Soils Engineer for the Oregon Highway Department and three years as Laboratory Director for Palmer & Baker, Engineers, Inc. of Mobile, Alabama. The former employment was interrupted by active duty as a first lieutenant with the Corps of Engineers in Korea from September, 1950 until August, 1951.

In 1955, Mr. Schaub was appointed Assistant Professor of Civil Engineering at Virginia Polytechnic Institute. In this position he was responsible for graduate and undergraduate instruction in soil mechanics and transportation engineering.

Mr. Schaub continued his graduate school education by attending Purdue University during the summers of 1956, 1957 and 1958. In September, 1958, he was granted leave from Virginia Polytechnic Institute and employed as an instructor in the School of Civil Engineering of Purdue University.

Mr. Schaub was married in June, 1948, to the former Malinda Bailey of Radford, Virginia.

Mr. Schaub is a member of the Society of Sigma Xi (Associate), American Society of Civil Engineers (Member), and the Highway Research Board (Associate). He is a registered professional engineer in Virginia. He is a member of Phi Kappa Phi, Tau Beta Pi, Chi Epsilon, and Kappa Sigma fraternities.

His publications are:

Schaub, J. H., "Soils Exploration and Design Considerations for the Greater New Orleans Expressway. Part I - Exploration and Testing," Proceedings, Highway Research Board, Vol. 35, 1956.

Goetz, W. H. and J. H. Schaub, "Triaxial Testing of Bituminous Mixtures," Special Technical Publication 252, American Society for Testing Materials, 1959.